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ERRATA.

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Page 55: Coulomb's Formula, on this page, should be changed to read as follows:

$$P = W h \times \tan.^2 \left(45^\circ - \frac{\text{angle of repose}}{2}\right).$$

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Page 493, lines 21 and 22: For "Owen Creek" read "Queen Creek."
Page 508, line 3 from bottom: Take out the words "forests are" and insert in their place the words "deforestation is."
Page 509, line 9 from bottom: Insert the word "million" after the word "thousand," so that the sentence will read: "A little while ago the country was told that a thousand million tons of soil are yearly washing from our agricultural lands into the sea."
Page 512, line 8 from bottom: Take out the word "not."
Page 540, line 19: Take out the words "considered in the abstract."

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1099

FOUNDATIONS FOR THE NEW SINGER BUILDING, NEW YORK CITY.*

By T. Kennard Thomson, M. Am. Soc. C. E.

With Discussion by Messrs. O. F. Semsch, Eugene W. Stern, Edwin S. Jarrett, and T. Kennard Thomson.

In August, 1906, the Singer Manufacturing Company awarded a contract to The Foundation Company of New York City, for sinking the pneumatic foundations of the addition to the old building on the northwest corner of Broadway and Liberty Street, now known as the Singer Tower.

This contract was for a lump sum, on the basis of the foundations being carried down to 70 ft. below the curb, or approximately to the average depth of the top of the hardpan as shown by the borings; for so much per cubic yard for everything between the depths of 70 and 75 ft.; and for an additional price per cubic yard for everything below 75 ft. The price to be deducted in case the depth of 70 ft. was not reached was about one-half of the price to be added. The progress reports of each caisson, given in Table 2, Plates IX and X, will show the justice of this proportion. The contract stipulated that the work should be completed in 110 days on the same basis of depths.

* Presented at the meeting of November 18th, 1908.

This is not only the fairest method of letting caisson work, but it is the only method which should be adopted; for if the contract is made for a lump sum to bed-rock, either the contractor must bid excessively high to allow for uncertainties, or run a good chance of losing his anticipated profits and much more besides. This is especially true where the borings are what are known as wash borings, which scarcely ever reach rock, but stop at the hardpan. This statement applies at least to wash borings made for sky-scrapers, although it does not apply to some which have been made for some of the railroads, where the borings have reached bed-rock by the use of an occasional small charge of dynamite.

Diamond-drill borings, of course, will find rock accurately, but only for the exact spot of the drill hole; for bed-rock here, known as New York gneiss, is very irregular, in fact it was found to be 6 ft. higher in one caisson than in another, although the two caissons were only 12 in. apart, each landing on a comparatively flat rock. In another case we landed one end of a caisson, 14 ft. long, on bed-rock and then had to dig 14 ft. deep at the other end to find rock. This is mentioned because the surface of the rock under the Singer Building is exceptionally level, as shown by the cross-sections of the caissons, Fig. 1.

The size of the lot is approximately 75 ft. on Broadway and 115 ft. deep on the south side.

The tower is square in cross-section and has 36 columns, all 12 ft. center to center, making it 60 ft. square—measuring from center to center of columns. These tower columns rest on 20 pneumatic caissons, as shown on Figs. 2 and 3, and are carried to bed-rock; Caisson No. 48-49, the second to be sunk, was carried to rock as an exploration caisson. The remaining caissons are not under the tower, and were stopped in good hardpan.

At first it was intended to stop all the caissons as soon as good hardpan was reached, but after several had been sunk, including Caisson No. 30-43, which is one of the tower caissons, it was decided that all those under the tower should be carried to bed-rock, and the question arose as to what could be done about Caisson No. 30-43, which had already been carried 7 ft. into the hardpan and filled with concrete. The contractors volunteered to tunnel under this caisson at

PLATE I.
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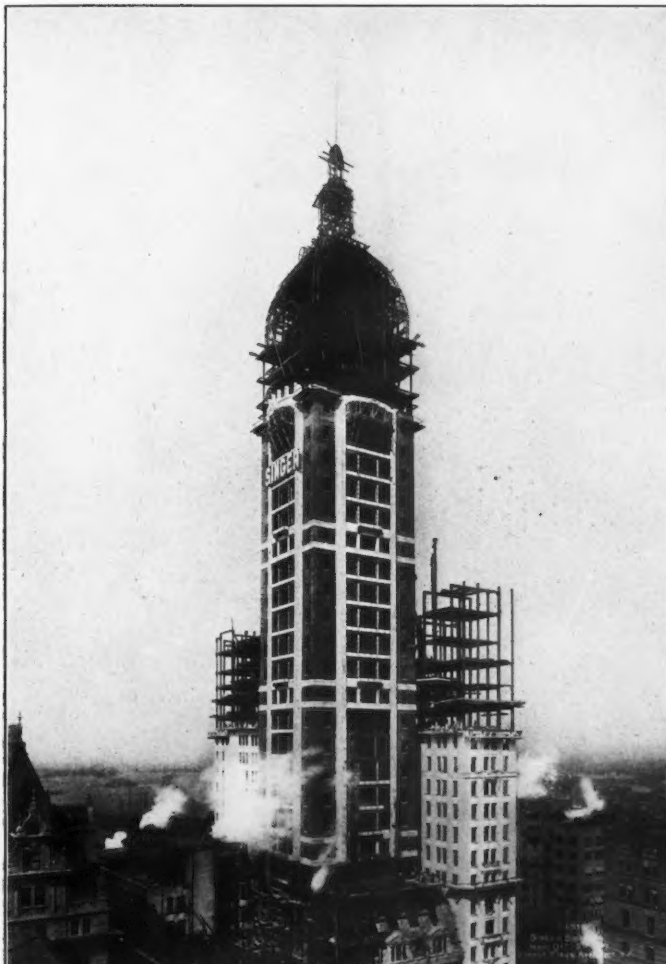


TOWER OF THE SINGER BUILDING DURING CONSTRUCTION.

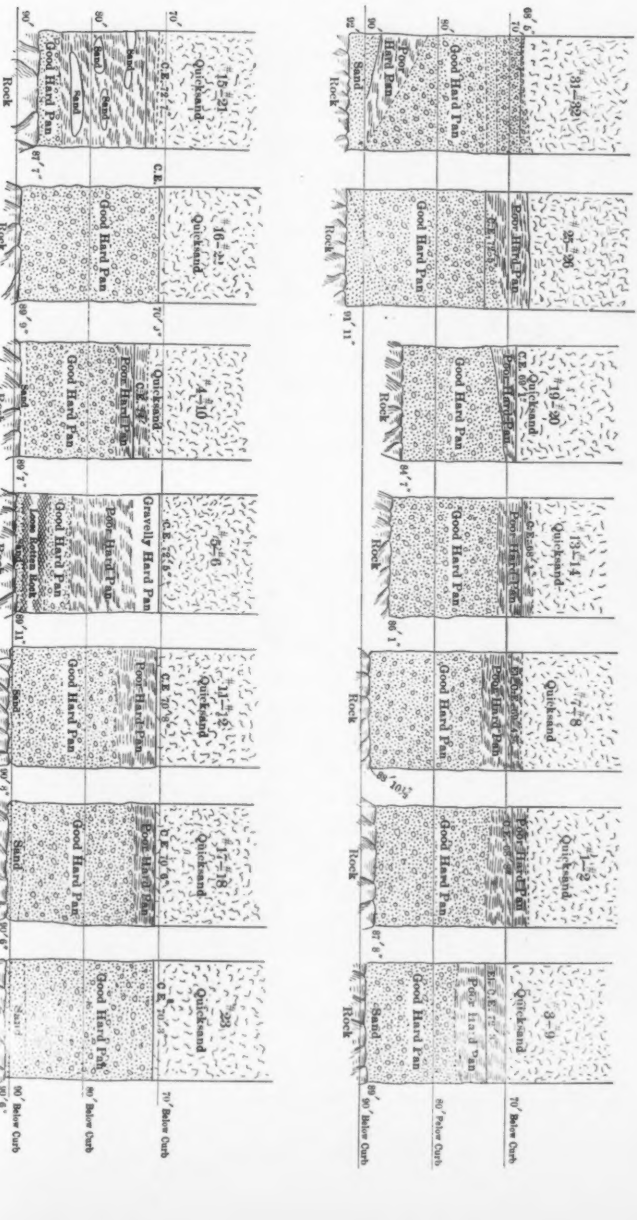

70' Below Curb
80' Below Curb
90' Below Curb
Quicksand
Poor Hard Pan
Good Hard Pan
Gravelly Hard Pan
Loose Rotten Rock
Sand
Rock

FIG. 1.

their own expense, so that the entire tower would rest on bed-rock, and this was done successfully as follows:

When Caisson No. 29-35 had been sunk some 5 ft. below the bottom of No. 30-43 (Fig. 4), a drift was cut under and to the north end of the latter. This drift was about 4 ft. wide and 5 ft. high, and when it had been completed all the hardpan under the north end of the caisson was removed (Fig. 5), the distance from the bottom of the old concrete to the top of the rock being 15 ft. at this point.

The hole under the north end was filled at once with concrete which was rammed up against the old base (Fig. 6). This, by the way, gave the opportunity to remove, from the original base, a piece of the concrete, put in under air, and this, having been made quite wet in the first place, was found to be very hard and compact. It is well known to caisson men that concrete when made properly with plenty of water sets very quickly in compressed air and becomes very hard.

As soon as the north end of Caisson No. 30-43 had been carried to rock, as before described, the adjoining caisson, No. 29-35, was excavated to rock, and the remaining hardpan under the south end of No. 30-43 was removed and the space concreted, thus completing, without any accident, probably what has been the only attempt ever made to undermine a pneumatic caisson (Fig. 7). Of course, if the original excavation had not been carried some 7 ft. below the cutting edge into good hardpan to which the concrete had firmly united, much more difficulty would have been encountered, if indeed the attempt had been made.

It might be stated that the cutting edge of a caisson is rarely carried much below the top of the hardpan, and it should not be, for, if the excavation is carried below the cutting edge into the hardpan and the space then filled with concrete, the concrete forms an excellent bond with the hardpan, greatly reducing the load on the base, which is usually from 12 to 15 tons per sq. ft. Whereas, if the caisson is carried through the hardpan there would be almost no friction below the top of the hardpan.

In this case the foundations were designed for 15 tons per sq. ft. In designing the foundations, it was decided that, if the wind pressure did not exceed 50% of the dead and live load, it would not be regarded, so the wind strains were not considered.

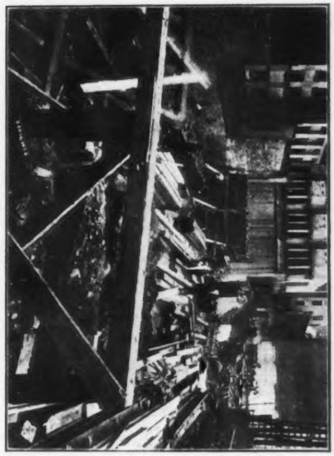
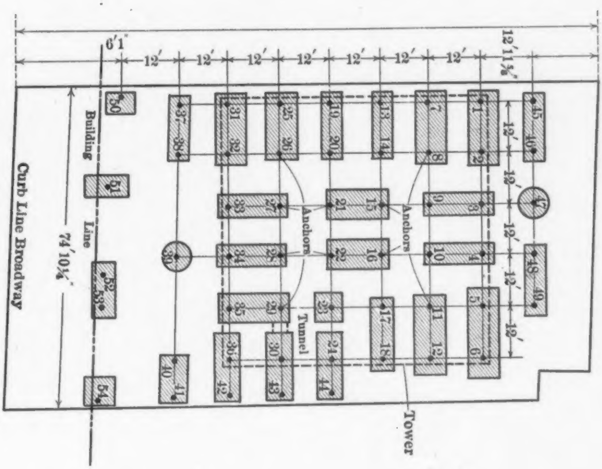


Fig. 1.—August 29th, 1906, Preparing to Build Platform.

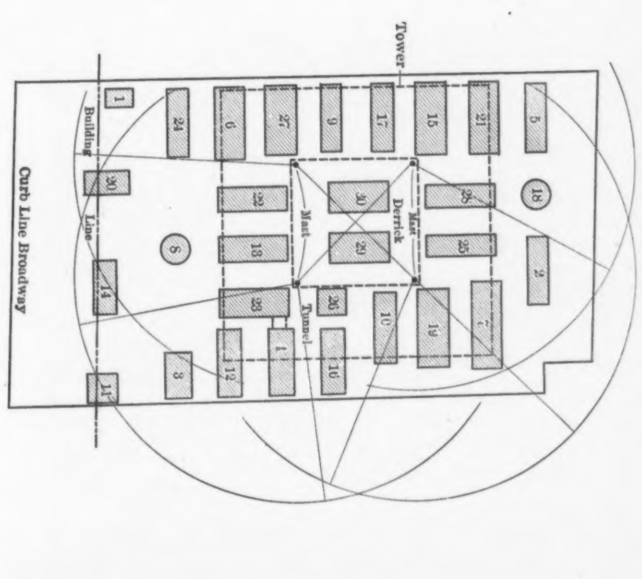
Fig. 2.—September 17th, 1906, Platform Partly Built; Erection of Derrick Commenced.

Fig. 3.—September 29th, 1906, Derrick Completed and in Use.

Fig. 4.—October 8th, 1906, Concrete Being Placed.



Location and Number of Columns, Caissons being designated by Column Numbers.
FIG. 2.



Order in which caissons were sunk.
FIG. 3.

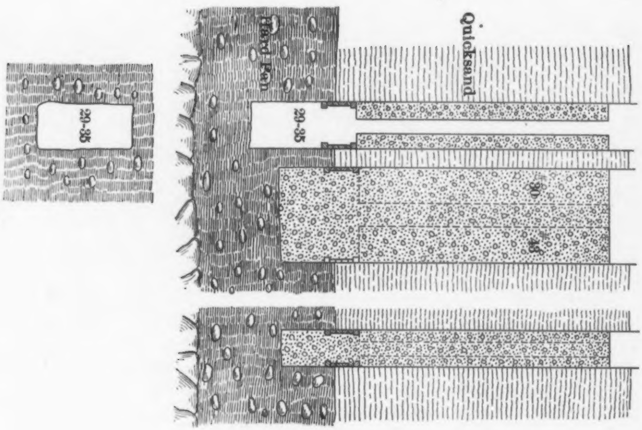

Quicksand
Hard Pan
20-35
30-43
20-35

FIG. 4.

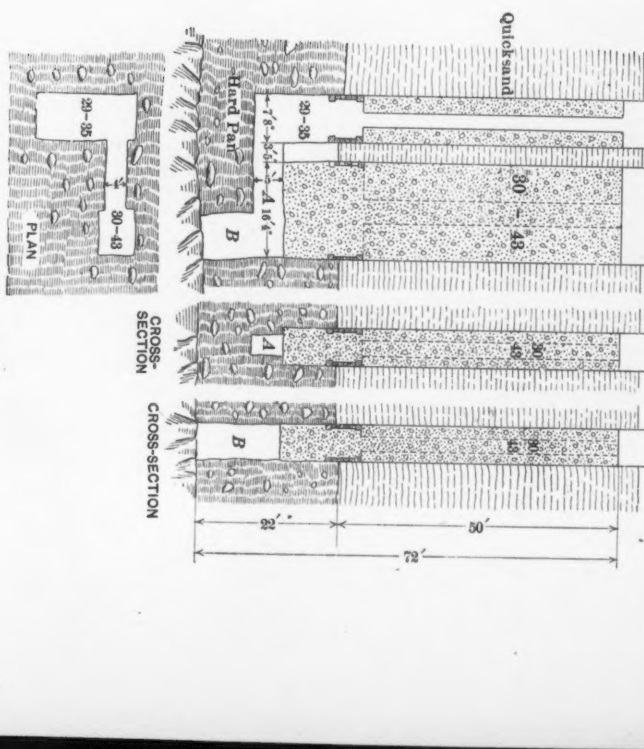

Quicksand
Hard Pan
20-35
7'8"
3'5"
A
16'4"
B
30-43
20-35
4'
30-43
PLAN
CROSS-SECTION
A
30-43
CROSS-SECTION
B
30-43
22'
50'
72'

FIG. 5.

PLATE III.
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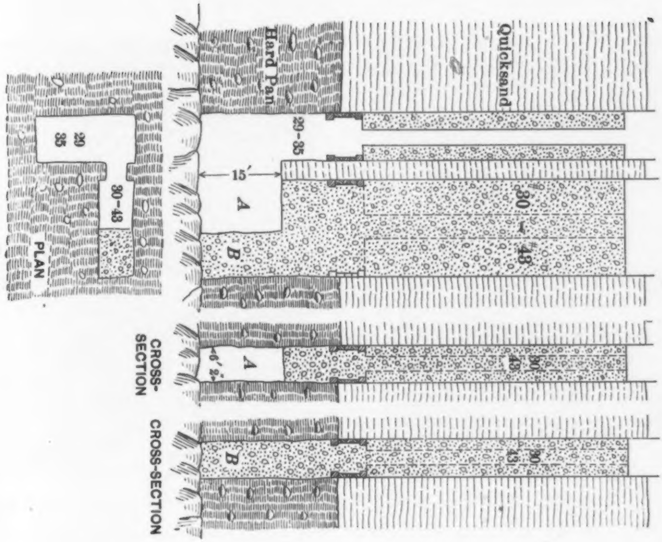
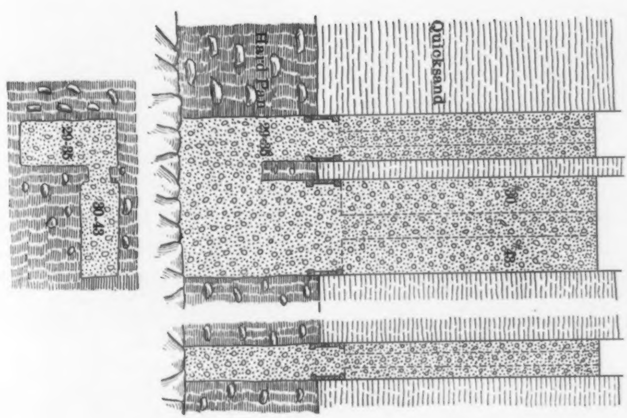
FIG. 1.—OCTOBER 18TH, 1906, TEMPORARY FORMS FOR CONCRETE ON TOP OF CAISSONS.

FIG. 2.—NEEDLE BEAMS SUPPORTING OLD BUILDING.

FIG. 3.—OCTOBER 18TH, 1906, SHOWING SEVERAL CAISSONS IN PROGRESS.

FIG. 4.—NOVEMBER 12TH, 1906, SHOWING PROGRESS.


Hard Pan
Quicksand
20-35
15′
A
B
30-43
20
35
30-43
PLAN
CROSS-SECTION
6′ 2″
A
30
43
CROSS-SECTION
B
30
43
Fig. 6.


Hard Pan
Quicksand
20-35
30
43
20-35
30-43
Fig. 7.

The following are some of the principal facts relating to the Singer Building:

Height of tower from bottom of caisson to top of flag-pole	745 ft.	
Height from basement floor to top of flag-pole	666 "	
Height from sidewalk to top of flag-pole	652 " 6 in.	
Height of tower from sidewalk to top of lantern	612 " 3 "	
Height of main building from sidewalk to roof	191 " 8 "	
Greatest depth of caisson below curb	92 "	
Area of each main floor	20 163 sq. ft.	
Area of each floor of tower	3 737 " "	
Total area of floor	411 333 " "	or 9.44 acres.

Plant.—The hoisting plant consisted of a four-boom derrick and two stiff-leg derricks, with five Lidgerwood double-drum engines, 3 ft. 7 in. by 10 in. and two, 8¼ by 10 in., and one Lambert, 7½ by 10 in.; in addition, there were four Rawson and Morrison boom-swinging gears.

A platform was built on the level of the sidewalk about 15 ft. above the excavation. It is customary to excavate the lot to about the water level before commencing the caisson work proper, and the derrick was built so that carts could run under it on the street level. It was about 30 ft. square, with four masts 30 ft. high and 50-ft. booms.

The compressor plant consisted of one Rand straight-line compressor with 14-in. steam cylinder, 18-in. air cylinder and 22-in. stroke, capable of pumping 1 294 cu. ft. of free air per minute, theoretical rated capacity, and one McKiernan 22-in. steam cylinder by 26-in. air cylinder by 24-in. stroke compressor, capable of pumping 1 474 cu. ft. of free air per minute with a speed of 100 rev. per min. Twin air receivers were used, each 41 in. in diameter, coupled, and 15 ft. 9 in. long. There was also a 14-in. air cooler, 14 ft. long.

The concrete was mixed in one Ransome mixer with a capacity of 24 cu. ft., and one Chicago mixer with a capacity of 18 cu. ft., both placed under the street platform.

FIG. 1.—NOVEMBER 8TH, 1906, SHOWING PROGRESS.

FIG. 2.—NOVEMBER 20TH, 1906, SHOWING PROGRESS.

FIG. 3.—NOVEMBER 30TH, 1906, BIRD'S-EYE VIEW.

FIG. 4.—COLLAPSIBLE SHAFTS AND BOTTOM-DUMP BUCKETS.

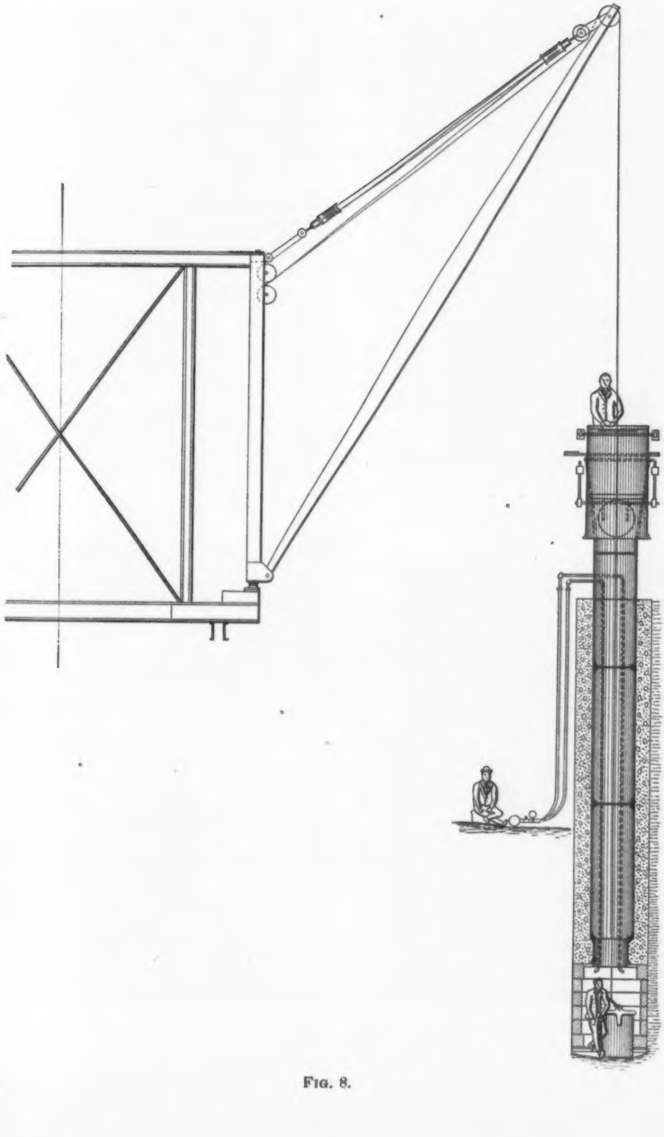


FIG. 8.

Fig. 1, Plate II, shows the ground on August 28th, 1906, with the contractors getting ready to build the platform. It will be noticed that they had already stored some 800 tons of pig iron on the site, ready for use, in addition to 400 tons of cast-iron blocks, each block weighing $1\frac{3}{4}$ tons, or as much as 70 or 80 pieces of pig iron.

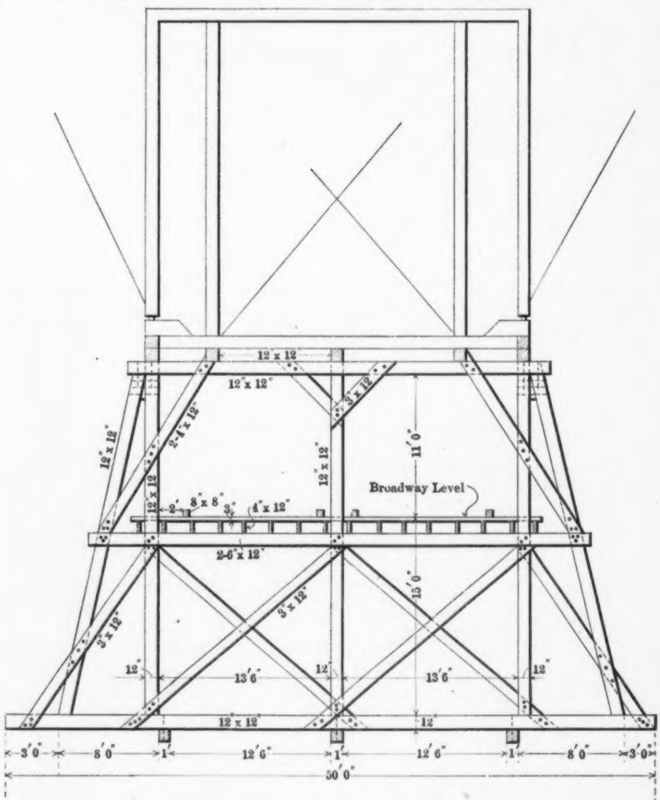


Fig. 9.

By September 17th, the platform had been partially built on the Broadway level and the erection of the four-masted derrick commenced. Fig. 2, Plate II, a view taken from Broadway, shows a temporary stiff-leg derrick with a gin-pole derrick in the rear used for the erection of the four-boom derrick.

PLATE V.
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FIG. 1.—COFFER-DAM ON TOP OF CAISSON FORMS.

FIG. 2.—JANUARY 21ST, 1907, REMOVING THE FOUR-MASTED DERRICK.

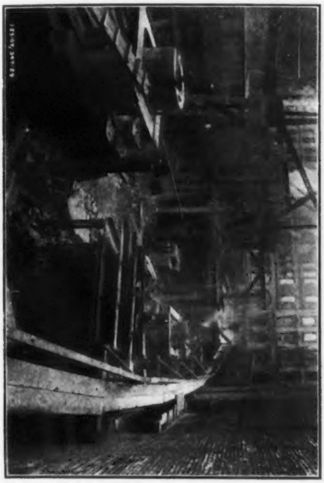

FIG. 3.—JANUARY 23D, 1907, LOOKING TOWARD BROADWAY, SHOWING PROGRESS.

FIG. 4.—JANUARY 23D, 1907, SHOWING PROGRESS.

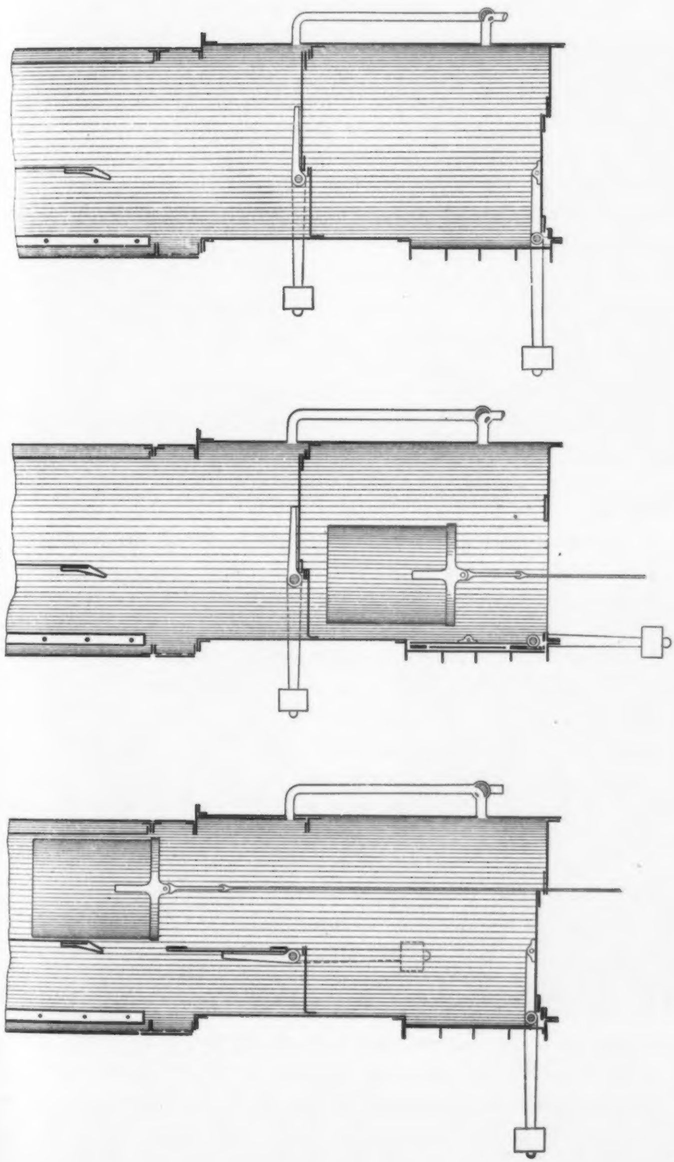

Fig. 10.

Fig. 3, Plate II, taken on September 29th, shows the four-boom derrick completed and in use, the two temporary derricks having been removed.

Fig. 4, Plate II, taken on October 8th, 1906, shows the caisson for Column No. 50, the first to be sunk into position, being concreted around the shaft which is covered up to prevent the concrete from falling into the working chamber. It also shows the caisson under Columns 40-41 to the right, in place, which was the third to be sunk, the second caisson being behind the derrick. It also shows another caisson resting on the I-beams on the platform.

Fig. 1, Plate III, taken on October 13th, 1906, gives a good idea of the temporary forms of 2-in. plank, tongued and grooved, and planed, used for the concrete on top of the caissons. As will be seen, the steel angles, usually 3 by 3 by $\frac{3}{8}$ in., are not cut to length for each size of caisson, but are allowed to project beyond the corners. It also shows the air-lock on the first caisson sunk—that under Column No. 50—the photograph having been taken two days before air was put on.

The four-masted traveler to the right is one of the traveling derricks on the lot of the City Investing Building, which was under construction at the same time as the Singer Building, the writer being retained on both. Some interesting problems were encountered, one of which was to locate the cutting edges of the caissons on each side of the lot line. Owing to the caissons not being strictly plumb, those of each building encroached on or under the adjoining property. Fortunately for the owners, both sides were trespassers, otherwise a lawsuit might have given much work to the lawyers and engineers.

Most specifications state very positively that a caisson shall not be out of plumb more than a certain amount, varying generally from 1 to 6 in.; but suppose a caisson is down 90 ft. in the ground and is found to be, say, 7 in. or more out of plumb—in fact, the writer has heard of, but has not seen, a caisson which was 5 ft. out of plumb—what can be done about it? Of course, a divergence of an amount like 5 ft. is absolutely inexcusable, but the best care will not prevent an error of a few inches. The only remedy is prevention—for an experienced man can start his caissons right and keep them so, and it is to the owner's advantage to attend to this from the start. If a caisson gets much out of plumb and down to a certain depth, it is

FIG. 1.—OCTOBER 26TH, 1906, SHOWING LACK OF SPACE.

FIG. 2.—CAISSON JUST LIFTED OFF TRUCK.

FIG. 3.—NEEDLE BEAMS AND SCREW-JACKS USED IN UNDERPINNING OLD BUILDINGS.

FIG. 4.—APRIL 5TH, 1907, STEELWORK BEING ERECTED.

not only impossible to plumb it again, but it is also impossible to prevent it from getting rapidly worse. The best superintendents rely on the engineers to keep them posted as to how the caisson is going, for the superintendent who tries to do this for himself with a hand level or plumb line will be much astonished to find how far he is out. This is the reverse in building masonry, for a good mason, when started right, can generally be relied on to build a pier absolutely to lines.

Fig. 2, Plate III, taken on October 13th, 1906, shows the needle-beams used to support the old building. As the underpinning has been described in the technical press, it will not be enlarged upon here.

Fig. 3, Plate III, taken on October 18th, shows the work in full blast; Caisson No. 50 to the left is taking a rest while the concrete is hardening in the forms. Caisson No. 48-49, the second to be sealed in under air, is shown in the rear, with heavy cast-iron weights piled on top of the concrete. It also shows Caisson No. 30-43 on the day when the compressed air was turned on, and Caisson No. 31-32, on which the lock is being placed. As has been stated, some 1 200 tons of cast-iron weights and pig iron were in use on this job at one time, and it can be readily understood what an expensive item it was. The great advantage of the cast-iron blocks, weighing 1¾ tons each, is the saving in time in handling. The only disadvantage is the necessity of using the derrick when it may be wanted for other purposes.

Fig. 1, Plate VI, gives a good idea of one of the serious difficulties which caisson men experience in city work, namely, lack of space; in fact, this is shown in nearly all the other plates. Here was a lock resting on the dumping platform, piles of temporary forms on the roadway, etc., and yet the contractors were obliged to be continually hauling plant, etc., back to the yard, to be returned when needed, perhaps in a few days.

Fig. 4, Plate III, taken on November 12th, 1906, shows where part of the dumping platform has been cut away for Caisson No. 28-34, the thirteenth to be sunk. It also shows 12 by 12-in. timbers piled up under the dump and a section of shafting in the corner, with forms everywhere—on top of the shanties, leaning against the wall, etc.

Fig. 2, Plate IV, taken on November 20th, shows a similar condition, the only space left clear being room for a team to drive from

Broadway, under the derrick, where it could be turned around by a good driver.

Fig. 3, Plate IV, a bird's-eye view, makes the unavoidable jumble look even worse.

Fig. 2, Plate VI, shows a caisson which has just been lifted off the truck, and gives an excellent idea of its construction. The bolts shown extend down to the cutting edge, and are 1 in. in diameter and about 3 ft. apart; the temporary 2-in. plank roof, on which the men are standing, was removed about 48 hours after 2 ft. of concrete had been placed upon it. Fig. 2, Plate VI, also shows the special provision made for "hooking on" to the caisson for hoisting, which is much neater and better than the old way—still often seen—of wrapping around the caisson a rope or chain which was always likely to slip and injure it.

The cutting edge is a 6 by 4 by ½-in. angle with the 6-in. leg horizontal; on top of the cutting edge is a horizontal course of 8 by 12-in. (12 in. vertical), then four courses of 6 by 12-in., on top of which is a 10 by 12-in. course. The longer caissons, of course, have struts and ties in the air-chamber.

Fig. 4, Plate IV, taken on January 7th, 1907, gives Broadway a rather unnaturally deserted appearance, but gives a good end view of the combination collapsible shafts, and under them some bottom-dump buckets.

Fig. 1, Plate V, taken on the same day, shows the coffer-dam on top of the forms of a caisson, but the writer can vouch for the fact that no caisson on this work was so much out of plumb as this picture would indicate; which goes to prove that the camera is sometimes quite a liar.

Fig. 2, Plate V, taken on January 21st, 1907, shows the four-masted derrick nearly all removed, the 28th caisson having been completed on January 18th, leaving only two to be done, and, as these were to be directly under the derrick, they could not be placed until the derrick and its platforms had been removed, so it was January 23d and 24th, respectively, before these two caissons were located, and the 13th and 19th of February before they were completed. Thus the work was done in 154 days from the time of signing the contract—the 110 days of the contract being understood to cover the time to be taken if the sinking stopped at Elevation 70; but all the caissons

PLATE VII.
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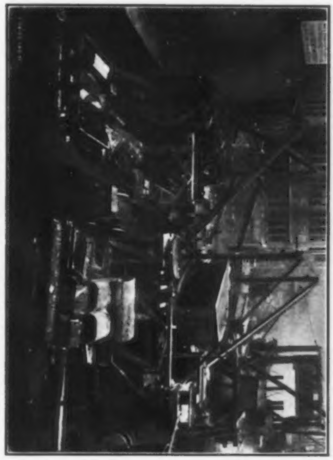


FIG. 1.—JANUARY 29TH, 1907, SPECIAL DERRICK FOR HANDLING THE LAST TWO CAISSONS.

FIG. 2.—FEBRUARY 16TH, 1907, SHOWING CONSTANT CHANGES IN PLATFORMS AS WORK PROGRESSED.

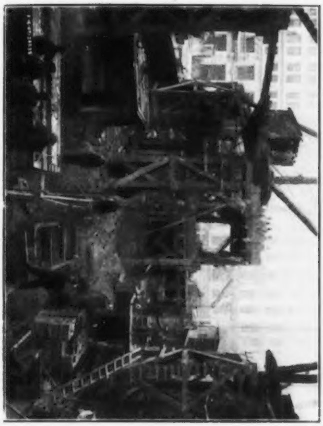


FIG. 3.—FEBRUARY 16TH, 1907, PLACING THE LAST CAISSON.

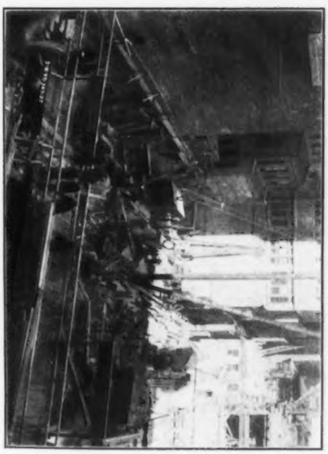


FIG. 4.—MARCH 16TH, 1907, STEELWORK COMMENCED.

went below this depth, many below 90 ft., and the additional 20 ft. of sinking consumed more time than the upper 70 ft.

Fig. 3, Plate V, was taken on January 23d, 1907, just before the 29th caisson reached the site. Caisson No. 3-9 (the 28th finished), though completed, still shows the lock and cast-iron weights in place. This view was taken looking toward Broadway, whereas the previous views were taken looking from Broadway.

Fig. 1, Plate VII, taken on January 29th, shows an additional derrick, erected on three high trestles, just for handling these last two caissons.

Fig. 3, Plate VI, shows the needle-beams and screw-jacks used in underpinning the old building; 40 and 60-ton screw-jacks were used.

Fig. 2, Plate VII, of February 16th, 1907, given a fair idea of the constant changes in the platforms, etc., as the work progressed from day to day.

Fig. 3, Plate VII, taken on February 16th, 1907, looking toward Broadway, shows the last caisson three days before the air was taken off. It also shows how rapidly the place was being cleaned up in order to set the bases, finish the concreting, etc.

One month later, that is, on March 16th, 1907, Fig. 4, Plate VII, shows that all the derricks of The Foundation Company have been removed, and that a big guy derrick for the iron erectors—Messrs. Milliken Brothers—has been put in place; the anchor-bolts, bases, etc., are everywhere in evidence.

Fig. 1, Plate VIII, of March 25th, shows two of these guy derricks in place, and Fig. 2, Plate VIII, and Fig. 4, Plate VI, of April 5th, show a very different scene, the erection of the steel-work being well under way. These two views show very plainly how the columns were anchored to the caissons, the detail of the anchors being shown on Fig. 11. Mr. O. F. Semsch designed these with sections decreasing from the top down to the bottom, at 60 ft. below the curb, the idea being to save the weight of the anchors by counting on the adhesion of the concrete to the steel bars, using 50 lb. per sq. in. for adhesion, as allowed by the New York Building Department.

Thus, for the bottom 10 ft., Mr. Semsch used one bar varying from 6 by 1½ in. to 6 by 3⅜ in.; for the next 10½ ft. above, he used two bars varying from 6 by 1¼ in. to 6 by 2½ in., and coupled to the lower

bar by pins 6½ in. in diameter; for the next 9½ ft., or from 40 ft. to 30½ ft. below the curb, he used three bars varying from 6 by 1 in. to 6 by 2⅜ in.; then four bars to 22 ft. below the curb, these running from 6 by 1 in. to 6 by 1¾ in. At this point he had a saddle with a 7¼-in. pin which connected the four flat bars with four round rods, which were 2¾ in. in diameter where the uplift was 270 tons, and 3⅝ in. in diameter where it was 480 tons. These are the round rods projecting above the column bases as shown in the plates. They were ordered 2 ft. longer than the calculated length so as to allow for any variation in the depth of the caisson, etc.

In three of the columns (Nos. 11, 16, and 22), the base was 8½ ft. lower, so the four top flat bars were omitted, the saddle being placed on top of the three flat bars.

The columns anchored were interior columns, ten in all, viz., Nos. 8, 11, 15, 16, 21, 22, 26, 27, 28, and 29.

Table 1, showing loads on foundations, is made up from figures of Messrs. Boller and Hodge.

By the courtesy of The Foundation Company, the writer was allowed access to all records of construction, and, therefore, has been able to give, in Table 2, the complete record of each individual caisson. This table shows the number of caissons on the site and the number under air at any one time. It also shows the difference in time consumed in going through quicksand and hardpan; for instance, in Caisson No. 17-18, the tenth to be sunk, 50½ ft. of sand were penetrated in 62 hours, while it took 91 hours to go through 20 ft. of hardpan. The writer has seen a caisson penetrate only 6 in. in 24 hours, where boulders, etc., were encountered.

It will be noticed that the total number of hours under compressed air is very much greater than the total number of hours of actual excavation and concreting. This was due to the fact that forms were used above the deck instead of coffer-dams, requiring the excavation to stop until the concrete was hard enough to allow the forms to be taken off safely.

Where coffer-dams are used, the sinking can be kept up continuously, even if the concrete is being placed below the ground level, which, of course, is impossible with forms which have to be removed. Again, where the penetration is thus suspended temporarily, the quick-

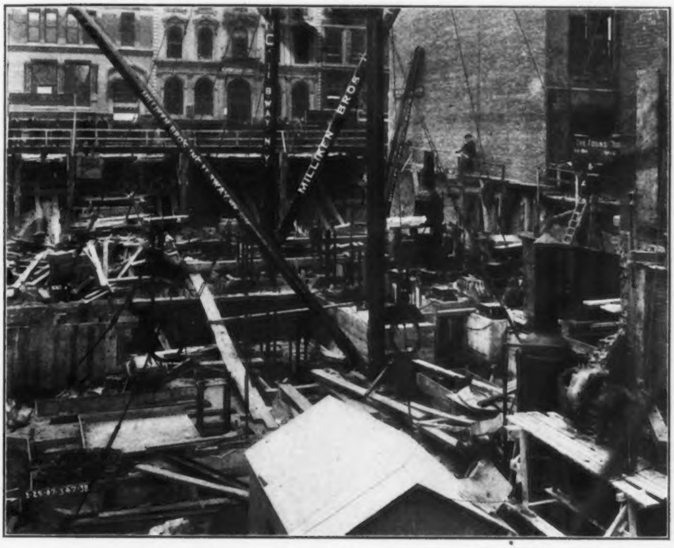


FIG. 1.—MARCH 25TH, 1907, TWO GUY DERRICKS READY TO ERECT STEELWORK.

FIG. 2.—APRIL 5TH, 1907, ERECTION OF STEELWORK WELL UNDER WAY.

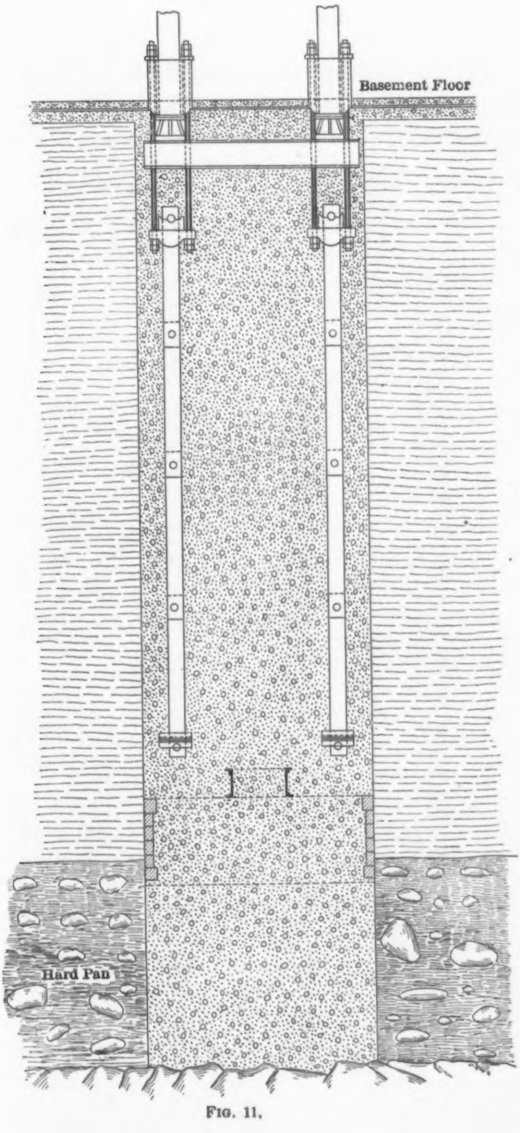


FIG. 11.

TABLE 1.—Loads on Column Bases, in Tons.

15 tons per sq. ft. allowed on caissons at top; no allowance for wind on caissons where wind load does not amount to 50% or more of the live and dead load.

Column number.	Dead load.	60% Live load.	Combined dead and live loads	Wind loads.
1	756.7	56.4	813.1	747.1
2	614.3	98.5	712.8	747.1
3	447.5	108.9	556.4	373.5
4	447.5	108.9	556.4	375.5
5	786.2	88.4	874.6	747.1
6	884.2	42.6	926.8	747.1
7	763.0	79.9	842.9	747.1
8	289.2	131.6	420.8	747.1 Anchor.
9	284.4	129.6	414.0	373.5
10	284.4	129.6	414.0	373.5
11	289.2	131.6	420.8	747.1 Anchor.
12	763.0	79.9	842.9	747.1
13	451.1	94.9	546.0	373.5
14	284.4	129.6	414.0	373.5
15	327.4	147.4	474.8	747.1 Anchor.
16	327.4	147.4	474.8	747.1 Anchor.
17	284.4	129.6	414.0	373.5
18	445.4	92.0	537.4	373.5
19	528.7	92.6	621.3	373.5
20	284.4	129.6	414.0	373.5
21	327.4	147.4	474.8	747.1 Anchor.
22	327.4	147.4	474.8	747.1 Anchor.
23	284.4	129.6	414.0	373.5
24	454.7	94.3	549.0	373.5
25	602.5	91.8	694.3	747.1
26	289.2	131.6	420.8	747.1 Anchor.
27	284.4	129.6	414.0	747.1 Anchor.
28	284.4	129.6	414.0	747.1 Anchor.
29	289.2	131.6	420.8	747.1 Anchor.
30	602.5	91.8	694.3	747.1
31	596.4	68.3	664.7	650.3
32	614.3	98.5	712.8	747.1
33	447.5	108.9	556.4	373.5
34	447.5	108.9	556.4	373.5
35	614.3	98.5	712.8	747.1
36	596.4	68.3	664.7	747.1
37	73.2	46.4	119.5	.....
38	133.3	85.1	218.4	.....
39	191.1	121.8	312.9	.....
40	131.8	83.9	215.7	.....
41	272.2	25.7	297.9	.....
42	214.4	18.8	233.2	.....
43	214.4	18.8	233.2	.....
44	142.8	9.4	152.2	.....
45	120.3	27.2	147.5	.....
46	143.6	43.2	186.8	.....
47	143.6	43.2	186.8	.....
48	143.6	43.2	186.8	.....
49	120.3	27.2	147.5	.....
50	275.9	36.9	312.8	.....
51	350.0	52.0	402.0	.....
52	242.4	37.9	280.3	.....
53	242.4	37.9	280.3	.....
54	328.5	29.8	358.3	.....

PLATE IX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXIII, No. 1099.
THOMSON ON
PNEUMATIC FOUNDATIONS.

TABLE 2.—RECORD OF SINKING CAISSONS FOR THE SINGER BUILDING, 1906-1907.

NUMBER OF CAISSON.	50	48-49	40-41	30-43	45-46	31-32	5-6	39	19-20	17-18	54	36-42	28-34	52-53	7-8
Order of sinking.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Size of caisson.	7' 0" × 7' 2"	5' 6" × 16' 4"	7' 2" × 11' 10"	6' 2" × 16' 4"	5' 6" × 16' 4"	7' 8" × 18' 4"	7' 8" × 21' 2"	6' 6" diam.	5' 6" × 17' 10"	5' 6" × 16' 8"	7' 0" × 8' 2"	6' 2" × 16' 4"	6' 2" × 16' 8"	5' 6" × 13' 8"	7' 8" × 18' 4"
Caisson arrived on lot { Hour Date.	5.30 P. M. Sept. 29	9.00 P. M. Oct. 11	4.20 P. M. Oct. 5	1.45 P. M. Oct. 6	4.30 P. M. Oct. 16	9.00 P. M. Oct. 6	6.30 P. M. Oct. 12	7.00 P. M. Oct. 26	9.30 P. M. Oct. 5	6.45 P. M. Oct. 9	5.30 P. M. Nov. 9	4.30 P. M. Oct. 8	11.25 A. M. Nov. 10	7.20 P. M. Nov. 13	8.00 P. M. Oct. 13
Compressed air on { Hour Date.	10.30 A. M. Oct. 15	1.20 P. M. Oct. 17	1.30 A. M. Oct. 16	5.00 P. M. Oct. 18	4.30 A. M. Oct. 27	1.00 A. M. Oct. 26	9.30 A. M. Oct. 29	6.00 P. M. Nov. 11	9.30 P. M. Oct. 31	9.00 P. M. Oct. 30	4.00 P. M. Nov. 16	12.30 P. M. Nov. 13	3.00 A. M. Nov. 18	7.00 A. M. Nov. 20	5.00 P. M. Nov. 12
Caisson reached hardpan { Hour Date.	2 A. M. Oct. 21	6.00 P. M. Oct. 21	10.00 A. M. Oct. 23	2.00 P. M. Oct. 27	9.30 A. M. Oct. 31	8.00 A. M. Nov. 4	10.30 A. M. Nov. 6	11.00 A. M. Nov. 13	10.00 P. M. Nov. 15	8.30 A. M. Oct. 31	7.00 P. M. Nov. 21	6.00 A. M. Nov. 23	11.00 P. M. Nov. 27	8.00 P. M. Nov. 30	1.30 A. M. Nov. 27
Excavation finished { Hour Date.	5.30 A. M. Oct. 22	9.00 P. M. Oct. 24	9.30 P. M. Oct. 24	2.00 P. M. Oct. 29	9.00 A. M. Nov. 2	6.00 A. M. Nov. 10	7.00 A. M. Nov. 11	11.00 A. M. Nov. 14	2.30 P. M. Nov. 19	7.30 P. M. Nov. 19	11.00 A. M. Nov. 23	11.30 A. M. Nov. 27	2.00 P. M. Dec. 1	2.00 P. M. Dec. 2	2.30 P. M. Dec. 2
Concreting begun in air-chamber { Hour Date.	6 A. M. Oct. 22	10.15 P. M. Oct. 24	10.00 A. M. Oct. 25	2.30 P. M. Oct. 29	10.00 A. M. Nov. 2	7.00 A. M. Nov. 10	9.30 A. M. Nov. 13	12.30 P. M. Nov. 14	3.00 P. M. Nov. 19	8.45 P. M. Nov. 19	11.30 A. M. Nov. 23	12.30 P. M. Nov. 27	2.30 P. M. Dec. 1	2.30 A. M. Dec. 2	3.00 P. M. Dec. 2
Compressed air taken off { Hour Date.	4 A. M. Oct. 23	7.00 A. M. Oct. 26	12.30 P. M. Oct. 26	2.00 P. M. Oct. 30	10.00 A. M. Nov. 3	8.00 A. M. Nov. 11	4.00 P. M. Nov. 14	6.00 A. M. Nov. 15	4.00 P. M. Nov. 21	8.00 P. M. Nov. 21	11.59 P. M. Nov. 23	10.30 A. M. Nov. 29	3.30 P. M. Dec. 3	4.00 A. M. Dec. 4	1.30 P. M. Dec. 5
Number of hours under compressed air	185½	209½	251	285	173½	391	390½	84	498½	527	176	382	372½	333	548½
Excavation stopped on	Hardpan	Rock	Hardpan	Hardpan	Hardpan	Rock	Rock	Hardpan	Rock	Rock	Hardpan	Rock	Rock	Hardpan	Rock
Ditching { Feet.. Hours	5 3	5 3	5 3	5 3	5 3	5 3	5 3	5 2	5 3	5 3	5 3	5 3	5 3	5 3	5 3
Excavation in quicksand { Feet.. Hours	40 70	51 67	47 77	49 62	44 55	44 70	50½ 82	46 41	43 66	50½ 62	46½ 50	49½ 54	50¼ 88	45 57	42½ 77
Excavation in hardpan { Feet.. Hours	10 27	13 69	11 35	7½ 31	11½ 36	25 142	19½ 117	82 24	16½ 82	20 91	12 40	21 102	16½ 87	14 42	21 132
Distance from curb to ground	21'	15'	15'	15'	18'	18'	15'	18'	20'	15'	15'	15'	15'	15'	20'
" " " " top of concrete	18' 6"	23' 1"	18' 8"	18' 8"	23' 1"	18' 8"	27' 2"	23' 1"	18' 8"	27' 2"	18' 8¾"	18' 8¾"	18' 8"	17' 5"	18' 8"
" " " " " " hardpan	64'	71' 4"	67'	69'	67'	67'	70' 5"	69'	68'	70' 6"	66' 5"	69' 6"	70' 3"	64' 11"	67' 10½"
" " " " bottom of excavation	74' 6"	84' 4"	77' 11"	76' 7"	78' 6"	92'	89' 11"	77' 5"	84' 7"	90' 6"	78' 5"	90' 6"	86' 9"	78' 11"	88' 10½"
" " " " cutting edge	66' 6"	73'	68' 5"	71' 4"	69' 6"	68' 6"	72' 5"	71'	69' 1"	70' 6"	68' 5"	69' 9"	70' 6"	67' 11"	69' 4½"
Number of feet of coffer-dam above concrete	0	0	4	4	0	0	14	0	0	14	4	4	7	4	4
" " cubic yards of excavation	99	244	197	229	218	385	449	73	234	256	135	282	273	177	358
" " " " " concrete	101	203	183	213	183	378	377	72	237	215	124	265	259	171	363

TABLE 2.—RECORD OF SINKING CAISSONS FOR T

Number of Caisson.		24–44	13–14	47	11–12	51	1–2	
Order of sinking.		16	17	18	19	20	21	
Size of caisson.		5' 6" × 16' 4"	5' 6" × 16' 10"	6' 6" diam.	7' 8" × 18' 2"	5' 6" × 11' 8"	7' 8" × 19' 4"	6'
Caisson arrived on lot	Hour	4.00 P. M.	3.00 P. M	7.00 P. M.	Noon	1.00 P. M.	11.45 A. M.	
	Date.	Oct. 9	Oct. 23	Oct. 26	Nov. 6	Nov. 28	Oct. 31	
Compressed air on	Hour	8.00 A. M.	5.00 A. M.	8.30 P. M.	8.00 A. M.	6.30 P. M.	Noon	
	Date.	Nov. 19	Nov. 21	Dec. 9	Dec. 3	Dec. 7	Dec. 3	
Caisson reached hardpan	Hour	8.00 A. M.	11.00 A. M.	8.00 A. M.	4.00 A. M.	9.30 A. M.	5.30 A. M.	
	Date.	Nov. 25	Dec. 6	Dec. 11	Dec. 7.	Dec. 15	Dec. 20	
Excavation finished	Hour	10.45 A. M.	10.00 P. M.	3.00 P. M.	7.00 A. M.	10.00 A. M.	11.00 P. M.	
	Date.	Dec. 8	Dec. 9	Dec. 12	Dec. 13	Dec. 17	Dec. 26	
Concreting begun in air-chamber	Hour	11.30 A. M.	10.30 P. M.	3.30 P. M.	7.30 A. M.	10.30 A. M.	11.30 P. M.	
	Date.	Dec. 8	Dec. 9	Dec. 12	Dec. 13	Dec. 17	Dec. 26	
Compressed air taken off	Hour	5.00 A. M.	1.30 P. M.	Noon	1.00 A. M.	5.30 A. M.	8.30 A. M.	
	Date.	Dec. 10	Dec. 11	Dec. 13	Dec. 15	Dec. 19	Dec. 29	
Number of hours under compressed air		501	488½	87½	281	274½	620½	
Excavation stopped on		Rock	Rock	Hardpan	Rock	Hardpan	Rock	
Ditching	Feet.	5	5	5	5	5	5	
	Hours	3	3	2	3	3	3	
Excavation in quicksand	Feet.	52	42	52	49	49	48	
	Hours	58	56	35	79	52	75	
Excavation in hardpan	Feet.	18.1	19	10	22	14½	21	
	Hours	98	83	31	140	48	130	
Distance from curb to ground		15'	20'	15'	15'	15'	20'	
" " " " top of concrete		27' 2"	18' 8"	23' 1"	27' 2"	19' 5"	18' 8"	
" " " " " " hardpan		72'	67'	72'	69'	69'	67'	
" " " " bottom of excavation.		90' 8"	86' 1"	82'	90' 8"	83' 6"	87' 8"	
" " " " cutting edge		72' 9"	68' 1"	73' 6"	70' 8"	70' 6"	68' 8"	
Number of feet of coffer-dam above concrete		14	4	12	18	4	4	
" " cubic yards of excavation		257	226	82	390	163	371	
" " " " " concrete		209	228	82	327	152	377	

OR THE SINGER BUILDING, 1906-1907 (*Continued*).

	27-33	29-35	37-38	4-10	23	25-26	3-9	16-22	15-21
	22	23	24	25	26	27	28	29	30
4"	6' 2" × 16' 8"	7' 8" × 17' 8"	5' 6" × 16' 2"	5' 6" × 16' 8"	7' 0" × 7' 0"	17' 8" × 17' 4"	5' 6" × 16' 8"	7' 0" × 17' 0"	7' 0" × 17' 0"
M.	3.00 P. M. Dec. 6	4.00 P. M. Nov. 24	3.20 P. M. Dec. 5	2.00 P. M. Dec. 19	8.00 P. M. Dec. 24	8.45 P. M. Dec. 16	7.00 P. M. Dec. 19	6.00 P. M. Jan. 23	Noon Jan. 24
	2.30 A. M. Dec. 14	10 A. M. Dec. 11	2 P. M. Dec. 15	3.00 P. M. Dec. 29	2.00 A. M. Jan. 4	12.30 P. M. Jan. 2	8.00 P. M. Dec. 30	Noon Feb. 5	12.30 A. M. Feb. 4
.)	10.30 P. M. Dec. 23	6 P. M. Dec. 21	4.00 A. M. Jan. 4	4.00 P. M. Jan. 6	7.00 A. M. Jan. 10	11.30 A. M. Jan. 10	2.00 P. M. Jan. 12	6.00 P. M. Feb. 8	10.00 P. M. Feb. 14
M. 6	2.30 P. M. Dec. 30	7 A. M. Jan. 6	4.00 P. M. Jan. 6	2.45 P. M. Jan. 9	7.00 A. M. Jan. 14	7.00 A. M. Jan 16	7.30 P. M. Jan. 15	11.30 P. M. Feb. 11	8.30 A. M. Feb. 18
M. 6	3.00 P. M. Dec. 30	7.30 A. M. Jan. 6	4.30 P. M. Jan. 6	3.20 P. M. Jan. 9	7.30 A. M. Jan. 14	7.30 A. M. Jan. 16	8.30 P. M. Jan. 15	11.45 P. M. Feb. 11	9.00 A. M. Feb. 18
M. 9	12.30 P. M. Jan. 3	12.30 A. M. Jan. 9	11.30 P. M. Jan. 7	6.30 A. M. Jan. 11	10.30 A. M. Jan. 15	10.30 A. M. Jan. 18	9.30 A. M. Jan. 18	4.30 P.M. Feb. 13	10.15 P. M. Feb. 19.
	490	686½	561½	303½	272½	382	445½	196½	381¼
:	Rock	Rock	Hardpan	Rock	Rock	Rock	Rock	Rock	Rock
	5 3	5 3	5 3	5 3	5 2	5 3	5 3	5 3	5 3
	50 77	50 96	43 53	51½ 57	49 43	41 64	51 58	50 74	50½ 67
	20 119	20 122	12 60	18 71	21½ 96	26 139	18 67	19 78	17 84
 ' ' '	15' 18' 8" 70' 90' 68' 10 285 271	15' 18' 8" 70' 2" 90' 2" 72' 10 377 358	20' 18' 8" 67' 9" 79' 9" 69' 9" 4 197 199	15' 27' 2" 71' 6" 89' 7" 74' 10 253 212	15' 27' 2" 69' 90' 6" 70' 3" 14 137 114	20' 18' 8" 66' 91' 11" 72' 5" 0 354 358	20' 23' 1" 71' 89' 72' 3" 14 251 224	15' 27' 2" 69' 9" 89' 9" 70' 9" 14 329 275	15' 18' 8" 70' 7" 87' 7" 72' 7" 4 320 304

sand is apt to pack against the caisson, greatly increasing the friction and requiring much more pig iron to overcome it.

This form of construction shows to the greatest advantage where it is possible to place all the concrete on the caisson before the sinking commences, or where the total depth of caisson and coffer-dam will not be more than 30 ft.

As soon as a caisson was placed and had some weight on it, it was "ditched," that is, sunk—generally about 5 ft.—without air, so as to make it safe to add the locks and more concrete and to make the bracing against overturning easier.

Instead of giving an average case or extreme cases, both of which are usually misleading, the history of every caisson is given in Table 2.

The writer has endeavored to confine this description to what was out of the ordinary, and the unique feature of the foundations of the Singer Building was certainly the undermining of Caisson No. 30-43. In tabulating No. 30-43, the results are those obtained when it was originally sunk, and do not include the tunneling.

Of course, a contractor always wants to stop pumping compressed air into a caisson as soon as he can do so with safety, after it has been filled with concrete; but it would be much better to keep up the air pressure for at least from 12 to 24 hours after the concreting is completed. The writer has repeatedly found that if this were done he could make the concrete in the air-chamber water-tight, whereas, if it were not done, the water would rapidly force its way through the concrete to the top, and nobody will claim that it is advisable to have water flowing through concrete before it has set.

It has been stated that the sample of concrete taken from the bottom of Caisson No. 30-43, when it was undermined, was of the hardest, showing that the concrete had set sufficiently before the air was taken off.

In Table 2, the caissons are designated by the number of the columns resting on them, as originally laid out on the column plan, Fig. 2.

The entire work was done under the personal direction of Mr. C. P. Coleman, the Executive Officer of the Singer Manufacturing Company, and Mr. Ernest Flagg, his Architect. Mr. O. F. Semsch was Chief Engineer for the Architect, and Mr. H. J. Howells had charge of the inspection for him. A. P. Boller and H. W. Hodge, Members, Am.

Soc. C. E., were the Consulting Engineers for the building, the Contractors being The Foundation Company, of which Mr. Franklin Remington is President, and D. E. Moran, M. Am. Soc. C. E., E. S. Jarrett, Assoc. M. Am. Soc. C. E., and L. L. Brown, M. Am. Soc. C. E., are members. Mr. Alexander Allaire was Superintendent in charge for the Contractors, and the writer was Consulting Engineer for the caisson work for the owners.

He desires to thank the owners, architects, engineers, and contractors for their kindness in furnishing information, records, etc., for this paper. It is unnecessary to state that their work on the building was well done.

DISCUSSION.

O. F. SEMSCH, ESQ.—Mr. Thomson states that some of the columns of the Singer Tower were anchored to the footings in order to provide for the uplift caused by the wind pressure on the building. A brief description of the wind bracing, and more particularly of the method of calculating the load due to wind pressure on the footings, may prove of interest. It was assumed that the tower would be exposed to a wind pressure of 30 lb. per sq. ft. from the top to the roof of the 14-story main building, the lower portion being sheltered from wind on all sides by the surrounding buildings. Mr. Semsch.

The wind moment equals 23.7% of the moment of stability, both moments being taken about the base of the tower. As the Building Code of New York City specifies that the wind moment of any structure shall not exceed 75% of its moment of stability, the design is well within the limits of the law.

There are eleven sets of braces to resist the wind pressure in a northerly and southerly direction, and ten braces in an easterly and westerly direction. These extend up to the thirty-third story; below the fourteenth story there are two additional braces in each direction. Each wind brace may be said to consist of a vertical truss 12 ft. wide and about 500 ft. high, formed by two lines of columns with cross-latticing between them.

On account of the unusual architectural treatment of the tower façades, it was impossible to install a system of bracing extending across the entire width of each side, which, of course, would have been the most natural thing to do. Each front of the tower consists of five equal bays, the three middle ones being combined into what is practically one huge window about 36 ft. wide. This window projects several feet beyond the wall line; it was impossible, therefore, to run any bracing across it. Accordingly, four braces were placed in each corner of the tower, forming really four small towers, each 12 ft. square, and the others were grouped around the elevator shafts in the center of the building.

Each brace was calculated so that it could stand alone and resist its share of the wind pressure. Thus, if ten of these braces were to be set side by side, with a plate, about 60 ft. wide by 500 ft. high, placed against them, they would be capable of resisting safely a pressure of 30 lb. per sq. ft. on that plate.

This method of calculating the wind load, according to what the speaker terms the "brace" system, to distinguish it from another method to be mentioned later, resulted in concentrating that load, as far as the foundations were concerned, under the sixteen columns at the four corners of the tower and under the elevator-shaft columns in the center. The designers, however, were sure that in reality the

Mr. Semsch. entire wind load would not be transmitted to the footings by these columns alone, but that some pressure would be carried down by each column of the tower, whether or not it formed part of the bracing. In order to get some idea of what this load for each column might be, the tower was regarded as a monolith, and a calculation was made of the distance the resultant of the wind load and the total weight of the tower would fall outside of the center at its base. This distance was found to be 4 ft. From this was ascertained the ratio according to which the average pressure under each column would be increased on account of this eccentricity. In this way the amount of wind load that each column would transmit to the footings was found, according to what might be termed the "monolith" system.

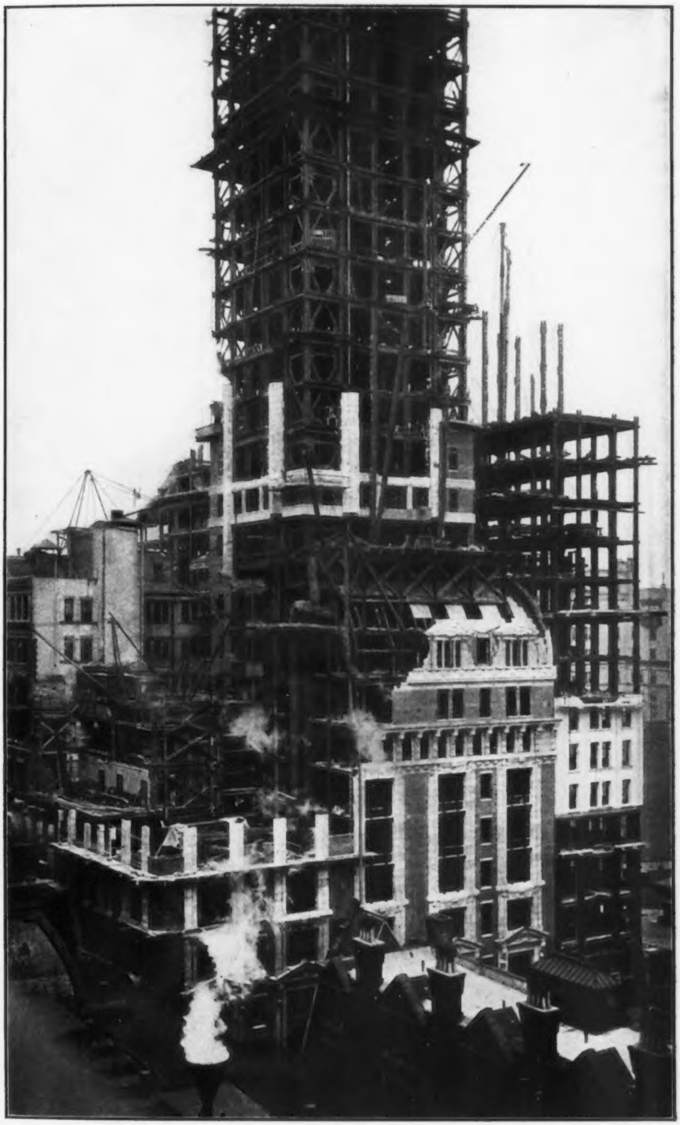
There were then two sets of calculations, and finally, in accordance with the suggestion of Mr. Ernest Flagg, the architect, the average between the loads obtained by each method was taken as the wind load on the footings, for it seemed reasonable that the correct solution of the problem lay somewhere between the two. In designing the anchors, however, the full uplift developed by the braces regarded as standing alone was taken.

Messrs. Boller and Hodge, who made a separate calculation, assumed that two-thirds of the entire wind load would be transmitted to the footings by the columns forming part of the wind braces, and the remaining third by the columns outside of the wind braces. Wherever the wind load at the base of a column amounted to less than 50% of the combined dead and live loads, they disregarded it altogether—according to the practice followed in bridge designing.

The wind loads given by Mr. Thomson in Table 1 were really reduced by one-third before being applied to the footings, because the Building Code of New York City allows an increase of 50% in stresses when calculating for wind load.

According to the architect's method of calculating, the total load on the tower footings proved to be, in round numbers, 30 000 tons. This, divided by the total caisson area, 2 794 sq. ft., gives an average pressure of 10.7 tons per sq. ft. The load is not evenly distributed, however, but varies from 15 tons per sq. ft. in some places to 5 tons in others. Messrs. Boller and Hodge allowed 15 tons per sq. ft. for live and dead loads, and 22½ tons per sq. ft. for live, dead, and wind loads. According to either method, the tower is amply safe, and its rigidity and solidity are really remarkable.

As mentioned by Mr. Thomson, several of the caissons were out of plumb a few inches, and one was out about 1 ft. Considering the depth, 90 ft., this was not to be wondered at. However, when the owner heard of it, he accepted the situation as soon as the Foundation Company had agreed to put a 12-in. bed of concrete over the entire area, and around the tops of the caissons, tying them all together.



THE SINGER BUILDING, NEW YORK CITY, DURING ERECTION.

Mr. Semsch.

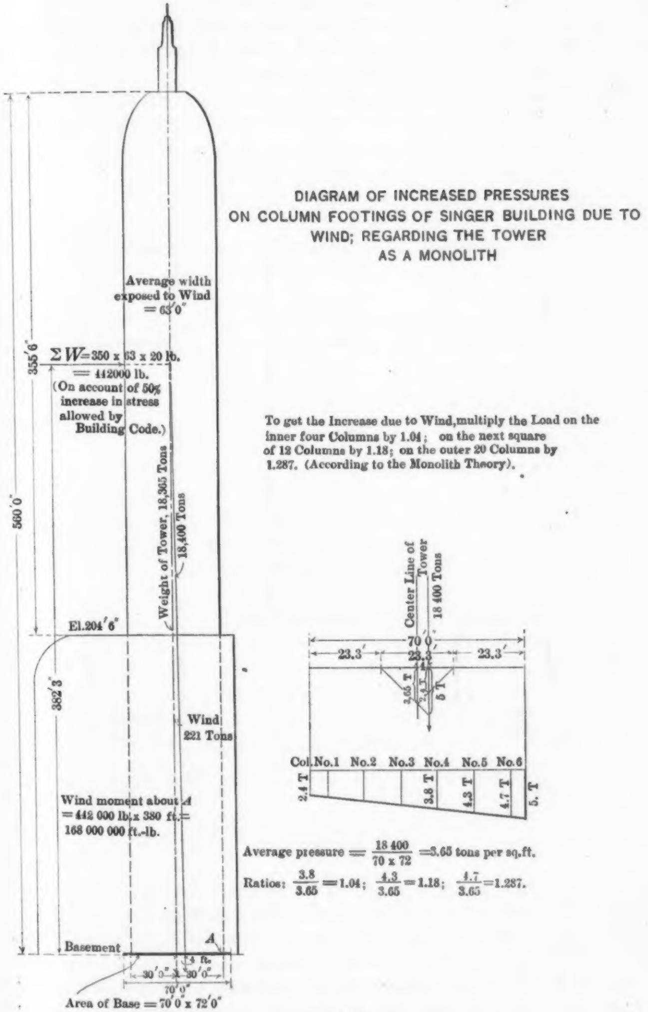
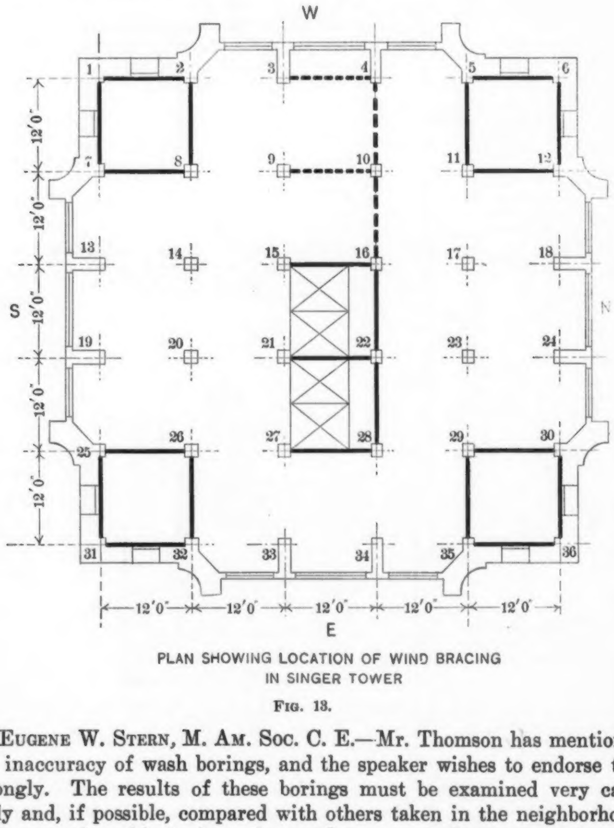


FIG. 12.

Mr. Semsch. This precaution was really not necessary, because the surrounding sand and hardpan held the caissons so securely in place that they could not have moved, but the owner very properly took the position that he was entitled to caissons which would be capable of standing up without any earth around them whatever.

This method of solving the difficulty may be used as a precedent by engineers if their clients ever refuse to accept caissons on account of their being out of plumb.



PLAN SHOWING LOCATION OF WIND BRACING IN SINGER TOWER

FIG. 13.

Mr. Stern. EUGENE W. STERN, M. AM. SOC. C. E.—Mr. Thomson has mentioned the inaccuracy of wash borings, and the speaker wishes to endorse this strongly. The results of these borings must be examined very carefully and, if possible, compared with others taken in the neighborhood, and even after this is done the results are not always to be trusted. In one very serious case, wash borings were taken, which showed rock 30 ft. below the surface, and the foundations were designed

accordingly. On sinking the foundations it was found that, instead of solid rock, the bottom consisted of broken stone filled in to a depth of 20 ft. on poor, soft bottom, the site having been originally a pond which was used as a convenient dumping ground for whatever was excavated in the neighborhood. Had this been known before the foundations were designed, many thousands of dollars would have been saved to the owners. Mr. Stern.

The method of placing a contract for foundations, namely, a lump sum to a certain depth, and an extra price per cubic yard for everything below that depth, commends itself also as being fair to both owner and contractor.

It would be interesting to know why it was decided to carry the foundation to rock after hardpan had been reached. Was not this considered sufficiently good foundation? The hardpan excavated in the neighborhood of the Singer Building is practically of the consistency of concrete. The speaker has had a specimen exposed in his office for months without the slightest sign of disintegration.

Mention is also made of the care required in sinking caissons in order to avoid tilting. This is, of course, very important, and too much care cannot be taken in this matter. It is money well spent. A dollar spent in avoiding misplacements will save hundreds later in trying to rectify them, and, in some cases, correction is impossible. It is necessary, not only that the work be started exactly right, but also that continual and careful observations be made with surveying instruments, in order that accuracy of position may be maintained.

The methods of sinking pneumatic foundations have been carried to a great degree of refinement in New York City, and perhaps nowhere else in any country has so much work of this kind been done, and so much experience been gained. The Moran air-lock is undoubtedly the greatest improvement that has been made in many years in connection with the actual processes in pneumatic work. There are some things, however, which the speaker's experience suggests as being capable of improvement, namely, that working chambers, which are usually made of wood, might better be made of reinforced concrete; and that the shafts connecting air-locks with working chambers are usually made too small to permit a man to climb the ladder in safety and avoid the hoisting bucket.

The speaker also agrees with Mr. Thomson that the process of sinking should be continuous, and that there should be no stoppages. If this is not done, the danger of the caisson being "hung up" is very great, and it is sometimes impossible to start the sinking again—even by loading it with cast-iron blocks—without blowing out the air; and this should never be done in building construction, as it is very likely to endanger the foundations of adjoining buildings.

Continuous sinking is not possible where the form method of

Mr. Stern. placing concrete is used rather than the coffer-dam method, and, while the latter method is perhaps a little more expensive in first cost, it is very questionable whether it is not cheaper in the end, for it enables the work to be done more accurately and quickly.

Mr. Jarrett. EDWIN S. JARRETT, ASSOC. M. AM. SOC. C. E.—The speaker disagrees in one particular with Mr. Stern. In so far as he criticizes adversely the practice, exemplified in the Singer Building foundation work, of moulding the concrete piers which surmount the caisson inside of temporary forms and of removing the forms before sinking the moulding piers, his views are not tenable. He states, in effect, that the long delays necessary at the various stages of the sinking operations to insure the setting of the concrete before it is safe to remove the forms and to continue the sinking have most injurious effects. The opportunity thus given for the increase of friction by the settling and compacting of the soil around the piers compels a recourse, as Mr. Stern views it, to excessive blowing in order to get the caissons to resume their downward movement. This blowing, in his opinion, very often throws the caissons out of plumb, and in general results in the loss of control of their movements. He favors a reversion to the old style of permanent coffer-dam forms because, in his belief, they allow a continuous sinking and thus insure, with reasonable care, complete control and better average results.

It may be stated at once that the continuous sinking of caissons is a partial insurance against unskilful methods and careless handling. On the other hand, with proper care and the skill acquired by long experience, caissons may be held up almost indefinitely and sinking may be resumed at any time without any bad results, it being premised, of course, that all the obvious precautions shall have been taken. The specific evil dwelt upon by Mr. Stern—the sinking of caissons out of plumb—results only from careless work. The presence or absence of a permanent coffer-dam form has little to do with it. If a structure is carried plumb to a depth of, say, 20 ft., it is not easy to throw it out of level. Only such excessive blowing as would allow material to flow in under the cutting edge could accomplish it, and such blowing is reckless and is not allowed by a careful contractor. If the caisson is carrying ample weight, it can be taken down plumb, even after a prolonged stoppage in the sinking. It is concluded, therefore, that the use of removable forms does not necessarily entail the bad work which Mr. Stern considers has resulted from their wide adoption.

Without discussing this particular matter any further, it may be stated that a small economy and an increased efficiency have resulted from this improvement in caisson work for buildings.

The construction and methods described by the author have been brought out somewhat fully in the technical journals. Mr. Thomson, however, has dwelt on the novel feature of this particular work, namely,

Mr. Jarrett.

tunneling through the hardpan from caisson to caisson and underpinning to rock a completed pier more than 50 ft. high. Aside from the interest of this operation, it has a significance which might, under conceivable circumstances, be full of possibilities. The ease with which tunnels, conduits, and the like could be carried through this hardpan as compared with the great difficulty of work, either in the quicksand above or in the rock below, might be of no small consequence. As a rule papers of this kind make no mention of certain emergencies which arise in such operations. Accidents occur in the progress of foundation work, and require great skill and nerve in handling, particularly when the work is being carried on close to heavy buildings. In such locations all the quicksand released will probably come from under the adjoining heavy structures.

It may be pertinent to mention an incident of this kind which occurred in the sinking of these foundations and shows the possibilities of damage, the great care which must be exercised, and the troubles that will arise in spite of that great care.

The steam for operating the compressors on this particular work was supplied by the New York Steam Company, the pipes of which are laid in the streets in the lower part of the city. All steam for such work in Lower New York City is thus supplied, as there is no available space upon which to erect boilers. It is necessary, of course, to keep the compressors in continuous operation, because there are always two or three caissons on the way down, and the loss of air would cause an inrush of quicksand, which it is absolutely necessary to prevent. To do this, the air pressure must be kept up, and of course the steam must be steadily supplied.

While this particular work was being done, the New York Steam Company's plant caught fire one night and, as the fire spread, boiler after boiler went out of commission. The air pressure began to decrease. There was one caisson about 15 or 20 ft. down, another more than half way down and a third was close to the hardpan. Had all the pressure been taken from the caissons without making provision to keep out the quicksand, it is extremely likely that great damage would have resulted. Having ascertained that all the steam pressure would be withdrawn, quick action was necessary. The only course open was to flood the caissons. Streams of water were thrown into each, and the working chambers, together with the shafts connecting them with the air-locks, were filled as rapidly as possible to a height above the water-level in the soil. There was thus obtained inside the caisson a compensating pressure against the water in the soil, which eliminated the danger that quicksand might enter.

After the air pressure was taken off, the caissons remained in the condition described, full of water, for from 24 to 36 hours. The compressors were then started, and air was applied to each caisson cau-

Mr. Jarrett. tiously, some of the water being blown out and the remainder being forced back into the soil by the air pressure. When the water subsided it was found that in no caisson had quicksand entered the working chamber, but it was in substantially as good condition as when the air pressure was taken off.

The flooding of caissons is resorted to quite often to drown out fires which in timber caissons are of not infrequent occurrence. The methods resorted to in order to meet such emergencies are the natural ones, but very often, in the middle of the night, with no one around to think and act quickly, and with no one on the ground who has been through a similar experience, such situations are hazardous.

Mr. Thomson. T. KENNARD THOMSON, M. AM. SOC. C. E. (by letter).—The writer regrets that more members have not accepted his invitation to discuss this paper, or at least to ask questions about it.

The Society is indebted to Mr. Semsch for his thorough discussion on the wind bracing of the Singer Tower.

The writer has been asked whether full air pressure was required in hardpan. Generally, when the cutting edge has entered a foot or so into good hardpan and has been well plastered with clay, it is possible to carry a little less pressure than the depth at the bottom of the excavation would require, but, if the hardpan is poor, or if a vein of sand is encountered, the full theoretical head is necessary.

Although some hardpan is very good, it is not all as good as concrete, nor anything like it, as may be judged from the typical cross-sections. When the owners of the Singer Building found that there were soft streaks in the hardpan they decided to sink all the tower caissons to rock and not run any chances of undermining by some future tunnel from Jersey City or Brooklyn. This was a very wise precaution, especially when the narrow base is considered.

Mr. Jarrett has mentioned the only proper thing to do when the air supply is cut off; the improper thing might also be mentioned: A few years ago the night superintendent on a certain foundation "lost his head" and pumped the water out of the air chamber and filled it up with sand, thus causing very serious settlement in an adjoining building.

In reference to the use of forms instead of coffer-dams, the writer is of the opinion that this is most economical where the depths to be sunk are so shallow that all the concrete can be placed before sinking starts; then there is no delay and no increased friction.

Reinforced concrete construction has been used for air chambers, but not for small caissons. Much progress has been made in caisson construction during the past few years, and is still being made. A firm which drops out of caisson work for three or four years will find old estimates of cost of very little use in bidding on new work, and when this is true of experienced men it is surprising to see the way

that novices underbid experienced contractors. It is certainly never to the advantage of an owner to let a contract to a company which has never had any experience in work of this kind, especially when the price bid is below actual cost. Mr. Thomson.

The writer has been asked how the "sand hogs" are paid, and in reply submits the following copy of the rates, terms, etc., as furnished by L. L. Brown, M. Am. Soc. C. E.

An agreement made and entered into by and between the firm of ———— and the International Compressed Air Workers of America, WITNESSETH:

First.—That from the first day of May, 1906, 8 hours constitutes a day's work, on Mondays, Tuesdays, Wednesdays, Thursdays, Fridays, and Saturdays of each week, including 30 min. for dinner.

Second.—That all labor performed on legal holidays, including Sundays, shall be entitled to an advance of 50%, whether in or out of the caisson.

Third.—That the minimum rate of wages for pressuremen shall be as follows: Up to 50 ft., $3.50 for one 8-hour watch, including 30 min. for dinner.

From	50	to	60 ft.,	$3.75	for two 3-hour watches,
"	60	to	70 ft.,	3.75	for two 2-hour watches,
"	70	to	80 ft.,	4.00	for two 1 hour and 30-min. watches,
"	80	to	90 ft.,	4.25	for two 1-hour watches,
"	90	to	95 ft.,	4.50	for two 45-min. watches,
"	95	to	100 ft.,	4.50	for two 40-min. watches.

From starting of concreting of air chamber, 50 cents extra per day shall be paid.

All depths to be measured from standard high tide, as established by the engineers of this port.

Fourth.—In case a gang is called out of the caisson, each man of the said gang be allowed full time for the said watch.

Fifth.—The minimum rate of wages for outside lock-tenders shall be $3.50 for 8 hours.

Sixth.—Foreman shall receive $1 per day extra, and assistant foreman shall receive not less than 50 cents per day extra.

Seventh.—That all employees shall be paid on Saturday of each week, up to and including the previous Thursday.

Eighth.—The minimum rate of wages for outside work shall be $3.50 for 8 hours.

Ninth.—That the firm of ———— agrees to employ only members of the I. C. A. W. U. of A., or such others as will be recognized by them throughout the United States.

Tenth.—That in case labor is required to weight down the caisson during the sinking, the members of the I. C. A. W. U. of A. in the employ of the firm of ———— shall have the preference.

Eleventh.—If any disputes arise, notice shall be given in writing by the party aggrieved within 24 hours after the same. Upon the failure of the party notified to adjust the said disputes, the same shall be submitted to arbitration.

Twelfth.—All disputes shall be submitted to a joint board of arbitra-

Mr. Thomson. tion, consisting of three persons selected by the firm of ——— with three members of the I. C. A. W. U. of A.; this board, failing to agree, shall select an umpire, whose decision shall be final and binding on both parties.

Thirteenth.—That a dressing-room with hot water, soap, towels, and coffee (made without steam) be furnished to the men on leaving the caisson; the temperature of said room to be regulated according to the weather. Also a day and night man to take charge of said room.

Fourteenth.—In case an employee is required to work outside, ample time will be given him to change his clothes after leaving the caisson.

Fifteenth.—That all foremen of the I. C. A. W. U. of A. shall have the privilege of hiring their own men.

Sixteenth.—That this agreement is to continue in force from the first day of May, 1906, until the first day of May, 1907, and if any change is contemplated by either party, notice in writing shall be given by the party desiring such change, at least three months prior to the expiration of this agreement.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1100.

THE LOW STAGE OF LAKES HURON AND MICHIGAN.

By C. E. Grunsky, M. Am. Soc. C. E.

With Discussion by Messrs. H. M. Chittenden, and C. E. Grunsky.

Lakes Huron and Michigan, if mean annual water elevation alone is considered, have been at less than the normal stage since 1888. These lakes were abnormally low in 1895 and in 1896. The Board of Engineers on Deep Waterways, in 1900, called attention to the depressed stage of the lake waters, giving the amount of the depression below what was then thought to be the normal at about 1 ft. for the preceding 15 years. At that time this depressed stage was attributed to certain natural and artificial changes that had been made at the head of St. Clair River, which is the outlet from the lakes.

The importance of restoring the lakes to normal elevation and of holding them so high that navigation interests will be fully protected, coupled with the desirability of withdrawing water from the lake system for sanitary and inland navigation purposes in limited, yet not inconsiderable, amounts has prompted the following study of the effect of water storage in Lake Superior upon the water elevation in Lakes Huron and Michigan, and of the causes to which the protracted low stages of the past in the two lower lakes should be ascribed.

The data herein used are taken from the published annual reports of the Chief of Engineers, U. S. Army.*

* See particularly the report of E. S. Wheeler, M. Am. Soc. C. E., Assistant Engineer, in Annual Report of the Chief of Engineers, U. S. A., 1903, Part 4, p. 2855.

Fig. 1 and Plate XII show the mean annual discharge for each of the two rivers, the St. Mary's, which flows from Lake Superior into Lake Huron, and the St. Clair, which, as already stated, carries the outflow from Lakes Huron and Michigan, and also the water-yield of the drainage basins tributary to the lakes, and the mean annual elevations of Lake Superior and of Lakes Huron and Michigan.

With this information, mass-curves of the water-yield of Lake Superior drainage basin could be constructed, and the effect of storage in this lake upon the discharge through its outlet, the St. Mary's River, could be determined.

Some of the conclusions reached from these studies and from the records referred to are here briefly stated.

TABLE 1.—Lake Stages.

Monthly Means for Typical Years. Elevations in Feet Above Sea Level.*

Lake Superior.

Month.	1861.	1869.	1879.	1892.	1901.
Jan.	602.59	602.21	601.39	601.38	602.65
Feb.	602.26	601.97	600.99	601.02	602.38
Mch.	602.12	601.52	600.74	600.84	602.13
Apr	602.53	602.10	600.87	600.99	602.27
May	603.16	602.50	601.25	601.50	602.65
June	603.31	602.52	601.38	601.86	602.60
July	603.47	602.88	601.68	602.00	602.97
Aug.	603.43	603.34	601.71	602.01	603.19
Sept.	603.34	604.19	601.64	602.07	603.07
Oct.	603.37	603.67	601.66	601.96	603.14
Nov.	603.03	603.33	601.48	601.68	603.08
Dec.	602.65	602.68	601.08	601.40	602.70
Year	602.94	602.74	601.32	601.56	602.74

Lakes Huron and Michigan.

Month.	1861.	1872.	1876.	1886.	1895.
Jan	582.83	580.99	581.74	582.67	580.03
Feb.	582.78	580.79	581.72	582.74	579.91
Mch.	582.92	580.29	581.85	582.93	579.92
Apr.	582.89	580.71	582.12	583.22	580.02
May	582.94	581.11	582.73	583.55	580.18
June.	583.18	581.51	583.22	583.64	580.26
July	583.27	581.61	583.66	583.48	580.23
Aug.	583.19	581.58	583.60	583.33	580.14
Sept.	582.99	581.48	583.49	583.15	580.01
Oct.	582.67	581.36	583.09	583.02	579.74
Nov	582.55	581.06	582.94	582.73	579.33
Dec.	581.20	580.77	582.75	582.43	579.09
Year	582.86	581.10	582.74	583.07	579.90

* The elevations in Table 1 are not in perfect accord with the lake-stage diagram published by the United States Lake Survey. They are based on elevations published in official reports of 1903.

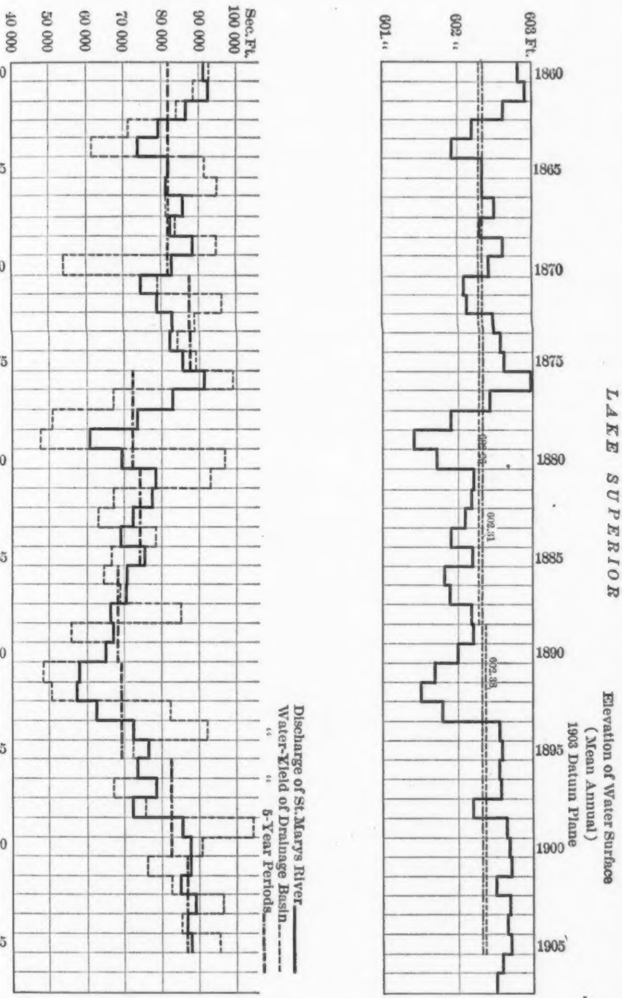

LAKE SUPERIOR
Elevation of Water Surface
(Mean Annual)
1903 Datum Plane
603 Ft.
602 "
601 "
602.31
602.38
Discharge of St. Marys River
Water-Yield of Drainage Basin
" " 5-Year Periods
Sec. Ft.
100 000
90 000
80 000
70 000
60 000
50 000
40 000
1860
1865
1870
1875
1880
1885
1890
1895
1900
1905

FIG. 1.

The area of the drainage basin of Lake Superior is about 76 100 sq. miles; the area of the lake is about 32 100 sq. miles.

The area of the drainage basin of Lakes Huron and Michigan, including the Lake Superior basin, is 213 900 sq. miles, and the water-surface area of the two lower lakes is 45 500 sq. miles.

TABLE 2.—DISCHARGE OF ST. MARY'S AND ST. CLAIR RIVERS.*
Computed from Published Monthly Means.

Year.	St. Mary's River.		St. Clair River.	
	Elevation, in feet, Lake Superior.	Mean discharge, in second-feet.	Mean elevation, in feet, Lakes Huron and Michigan.	Mean discharge, in second-feet.
1860	602.8	91 400	582.63	227 800
1861	602.9	92 200	582.59	226 800
1862	602.6	86 500	582.56	223 200
1863	602.2	79 900	582.11	213 800
1864	601.9	73 600	581.55	203 400
1865	602.3	81 600	581.29	196 900
1866	602.3	81 200	580.95	187 500
1867	602.5	85 800	581.41	198 400
1868	602.3	82 000	580.91	187 900
1869	602.6	88 400	581.03	190 500
1870	602.4	82 300	581.92	209 500
1871	602.1	74 600	581.94	214 100
1872	602.1	78 800	580.88	193 000
1873	602.5	82 500	581.35	198 800
1874	602.6	82 400	581.80	206 600
1875	602.6	85 200	581.52	205 700
1876	603.0	91 200	582.61	223 300
1877	602.4	82 800	582.40	218 200
1878	601.9	73 600	582.07	211 500
1879	601.4	60 600	581.17	191 500
1880	601.7	69 600	581.28	198 500
1881	602.2	78 300	581.79	205 000
1882	602.2	77 500	582.20	209 000
1883	602.1	73 300	582.43	215 800
1884	601.9	69 900	582.59	217 600
1885	602.2	75 300	582.79	226 800
1886	601.8	70 700	583.01	231 600
1887	601.9	70 200	582.37	217 200
1888	602.2	66 300	581.66	204 000
1889	602.2	67 500	581.21	192 500
1890	602.0	65 200	581.08	185 600
1891	601.7	58 300	580.48	176 400
1892	601.5	57 400	580.38	171 000
1893	601.8	63 000	580.62	177 700
1894	602.6	72 500	580.77	179 700
1895	602.6	76 800	579.78	164 400
1896	602.6	73 800	579.50	155 500
1897	602.6	78 900	580.12	171 100
1898	602.2	72 200	580.30	171 200
1899	602.7	85 700	580.30	174 000
1900	602.7	87 200	580.29	172 300
1901	602.7	87 200	580.55	180 100
1902	602.5	85 000	580.20	167 300
1903	602.7	89 000	580.34	172 500
1904	602.7	87 000	580.78	182 500
1905	602.7	88 000	580.91	185 000
Means	602.31	77 900	581.36	196 400

*The lake elevations in this table are in substantial conformity with the diagram issued by the U. S. Lake Survey. The discharges of St. Mary's and St. Clair Rivers are, for the most part, from the official reports already mentioned.

PLATE XII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXIII, No. 1100.
GRUNSKY ON
THE LOW STAGE OF LAKES HURON AND MICHIGAN.

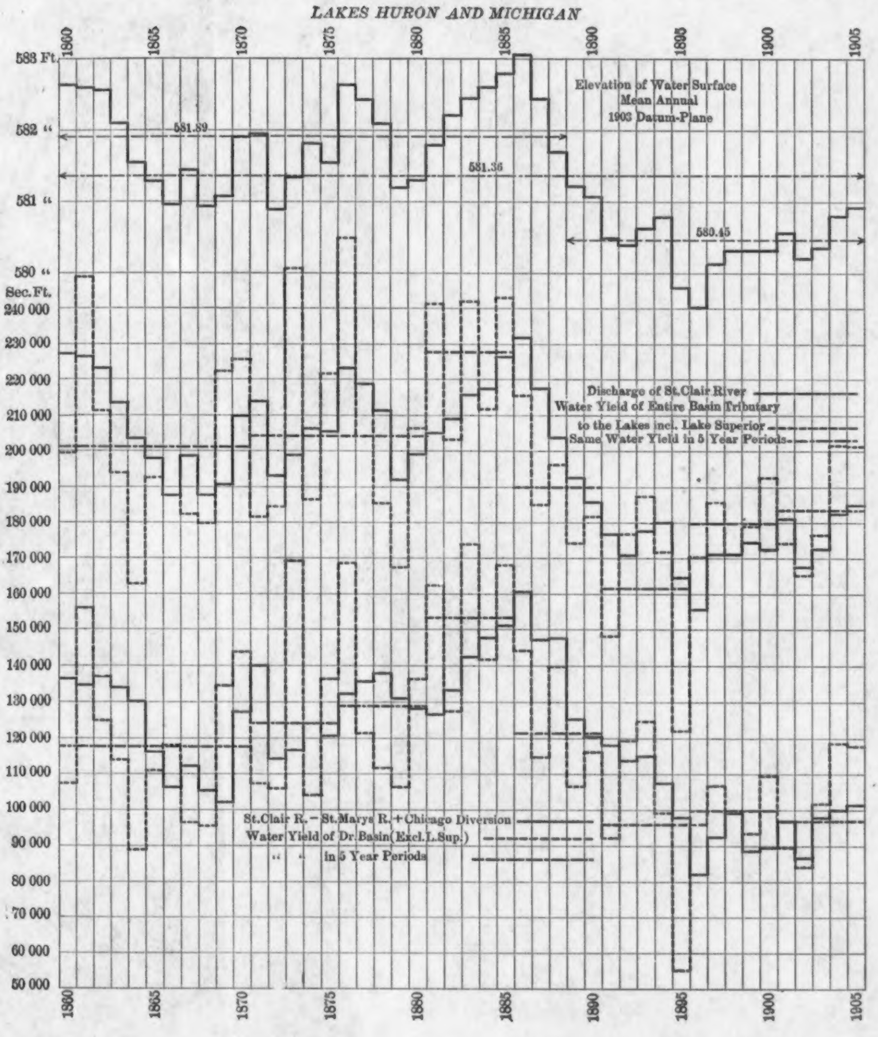


TABLE 3.—Lake Superior.

Elevation and Outflow Through St. Mary's River.

	1860-1888.		1889-1905.		1860-1905.	
	Elevation, in feet.	Outflow, in second-feet.	Elevation, in feet.	Outflow, in second-feet.	Elevation, in feet.	Outflow, in second-feet.
Lowest monthly mean...	600.75 Mar., '79 Mar. '80	49 500 Mar., '79	601.00 Feb., '93	44 700 Mar.,'92	600.75	44 700
Highest monthly mean...	604.15 Sep., '69	116 600 Sep., '69	603.50 Sep., '99 Oct.,1900	100 400 Sep., '99	604.15	116 600
Range of monthly means..	3.40	67 100	2.50	55 700	3.40	71 900
Lowest annual mean.....	601.35 1879	60 600 1879	601.55 1892	57 400 1892	601.35	57 400
Highest annual mean....	603.00 1876	92 200 1861	602.75 1901	87 200 1901	603.00	92 200
Range of annual means....	1.65	31 600	1.18	29 800	1.65	34 800
Minimum seasonal range of monthly means, low to succeeding high......	0.61 1870	......	0.68 1891	......	0.61	......
Maximum seasonal range of monthly means, low to succeeding high.....	2.67 1869	......	1.84 1899	......	2.67	......
Ordinary seasonal range of monthly means	1.28	......	1.24	......	1.26	......
Mean, whole period.........	602.26	78 900	602.38	76 100	602.31	77 900

TABLE 4.—Lake Superior.

Elevation and Outflow Through St. Mary's River, and Water-Yield of Drainage Basin for Periods of About Five Years.

Period.	Mean elevation, in feet.	Mean discharge of St. Mary's River, in second-feet.	Mean water-yield of drainage basin, in second-feet.
1860-65.................	602.45	84 200	81 400
1866-70.................	602.42	83 900	81 400
1871-75.................	602.38	80 700	87 100
1876-80.................	602.08	75 600	72 300
1881-85.................	602.12	74 800	74 000
1886-90.................	602.02	68 000	68 000
1891-95.................	602.04	65 600	69 300
1896-1900...............	602.56	79 600	82 100
1901-05.................	602.66	87 200	86 900
Means.................	602.33	77 900	78 100

All elevations herein noted are based on the precise levels of 1903.

The area of the drainage basin of Lakes Huron and Michigan is given as 137 800 sq. miles. The water surface of the two lakes has an area of 45 500 sq. miles.

TABLE 5.—LAKES HURON AND MICHIGAN.

Elevation and Outflow Through St. Clair River.

	1860–1883.		1889–1905.		1860–1905.	
	Elevation, in feet.	Outflow, in second-feet.	Elevation in feet.	Outflow, in second-feet.	Elevation, in feet.	Outflow, in second-feet.
Lowest monthly mean.	580.05 Mar., '69	135 200 Feb., '72	579.00 Dec., '95	110 600 Feb. '96	579.00	110 600
Highest monthly mean.......	583.60 Jun., '86	272 400 Jun., '86	581.80 July, '89	222 000 Aug.'89	583.60	272 400
Range of monthly means......	3.60	138 000	2.80	129 500	4.60	161 800
Lowest annual mean.........	580.88 1872	187 900 1868	579.50 1896	155 500 1896	579.50	155 500
Highest annual mean	583.01 1886	231 600 1886	581.21 1889	192 500 1888	582.95	231 600
Range of annual means.......	2.13	43 700	1.71	37 000	3.45	76 100
Minimum seasonal range of monthly means, low to succeeding high..........	0.34 1879	......	0.35 1895	......	0.34	......
Maximum seasonal range of monthly means, low to succeeding high................	2.10 1876	......	1.55 1899	......	2.10	......
Ordinary seasonal range of monthly means...............	1.06	......	1.09	......	1.08	......
Mean, whole period.............	581.91	208 800	580.52	174 100	581.36	196 400

In giving the seasonal range of monthly means in Table 5, the rise from a low to the following high stage alone was taken into account. In the season 1871-72, there was a drop in the water surface of 2.58 ft.

The water-yield of the lake basins in the foregoing tables was determined as follows: In the case of Lake Superior, the annual water-yield is the outflow from the lake, that is, the discharge of St. Mary's River increased by the storage increase, or decreased by the storage decrease, in Lake Superior during the year. The annual water-yield of the entire drainage basin of Lakes Huron and Michigan

(including Lake Superior) is the flow of St. Clair River plus the discharge through the Chicago Drainage Canal, increased by the annual storage increase in these two lakes, or decreased by the storage decrease. The water-yield of the restricted drainage basin of Lakes Huron and Michigan, that is of the drainage basin exclusive of Lake Superior, is found by subtracting the mean annual discharge of St. Mary's River from that of St. Clair River and adding to the remainder the annual storage increase in Lakes Huron and Michigan or subtracting therefrom the annual decrease of storage, as the case may be, and also adding the amount of water diverted into the Chicago Drainage Canal (which was opened in January, 1900, and takes about 4 167 sec-ft. of water from Lake Michigan).

TABLE 6.—LAKES HURON AND MICHIGAN.

Elevation and Outflow Through St. Clair River and Water-Yield of Drainage Basin for Periods of About Five Years.

Period.	Mean elevation, in feet.	Mean discharge of St. Clair River, in second-feet.	Mean water-yield* of drainage basin (excl. Lake Superior), in second-feet.	Mean water-yield* of drainage basin (incl. Lake Superior), in second-feet.
1860-65	582.14	215 300	117 000	201 800
1866-70	581.25	194 800	118 100	202 000
1870-75	581.49	203 600	124 500	205 200
1876-80	581.72	208 600	129 000	204 600
1881-85	582.37	214 800	153 300	228 100
1886-90	581.87	206 200	121 700	189 700
1891-95	580.39	173 400	95 800	161 400
1896-1900	580.12	168 800	99 700	179 300
1901-05	580.56	178 300	96 500	183 700
Means	581.36	196 400	117 400	195 200

*The term, water-yield, as here used, means the total delivery of water into the two lakes in excess of evaporation from the lake surface.

The normal outflow from Lake Superior through St. Mary's River, as determined from the records covering the 46 years, 1860 to 1905, is 77 900 sec-ft. The mean flow of the river during the period, 1860 to 1888, was 78 900 sec-ft., and, from 1889 to 1905, it was 76 100 sec-ft. The monthly mean flow of St. Mary's River ranges from about 45 000 to about 117 000 sec-ft.

If this lake were converted into a storage reservoir by the construction of works for the regulation of the flow of St. Mary's River,

it would be possible to hold the lake at or near a high stage until the stored water is needed to supply a deficiency in Lakes Huron and Michigan. The degree of benefit that can thus be secured will depend obviously upon the amount of water that can be stored, that is, upon the permissible range from low to high stage of Lake Superior and upon the capacity of the outlet channels. The records for the 46 years, 1860-1905, indicate a range of mean monthly lake stages of 3.40 ft., and a range of the annual means of 1.65 ft. The range of monthly means from low to high in single seasons is normally 1.26 ft., but this amount is frequently exceeded, and there is one season noted, 1869, in which it reached 2.67 ft. This was an unusual fluctuation, and strikingly illustrates the fact that there may be a material departure at certain times each year from the mean annual stage. The probable and possible departure should be carefully studied when works for the complete control of the outflow from the lakes are planned.

A description of the works for the partial control of lake stages, made necessary by the construction of the Lake Superior Power Canal, which have been in service since 1902, will be found in *Transactions*.*

By computation in the usual way, with recourse to the mass-curve of the annual water-yield of the Lake Superior basin, it can be shown that, under complete regulation, with a range 1.5 ft. between the extremes of mean annual lake elevations, there would be material improvement over natural outflow conditions. This amount of storage is equivalent to a flow of 42 600 sec-ft. for 1 year. In a succession of seasons such as those from 1860 to 1888, it would keep the outflow at a minimum annual mean of about 69 200 sec-ft. The outflow of 60 600 sec-ft. in 1879 could have been increased by 8 600 sec-ft. From 1860 to 1888, there would have been no time when the mean annual delivery of water from Lake Superior into Lakes Huron and Michigan would have been less than the above indicated minimum of 69 200 sec-ft., unless by intent to conserve water for a subsequent year.

An examination of the mass-curve of water-yield for the entire period, 1860 to 1905, shows the real critical period to have been from 1888 to 1893, in the last three years of which the discharge of St. Mary's River fell to 58 300, 57 400, and 63 000 sec-ft., respectively. A controlled storage capacity of 1.5 ft. in depth over Lake Superior

* "The Compensating Works of the Lake Superior Power Company," by G. F. Stickney, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIV, p. 346.

would have made it possible to increase these amounts to a constant flow of 65 700 sec-ft.

On the assumption that the two critical periods—the one including the years 1878 and 1879 in the first 20 years of the discharge record, and the last-mentioned period—indicate what may be expected in the future, it may be broadly stated that an available storage of 1.5 ft. in depth over Lake Superior would enable the lake outflow to be maintained above a minimum annual mean approximating 65 700 sec-ft. Should it be found practicable to make the effective storage in Lake Superior 2.5 ft. between the extremes of annual mean stages, then the regulation of the discharge from the lake will make it possible to keep the minimum mean annual outflow higher than if storage be restricted to only 1.5 ft. It is found in this case, if past records be again examined, that the critical period in the last 48 years would have extended from 1876 to 1893. In this period of 16 years the regulation of outflow and the addition of stored water would have raised the minimum mean annual discharge from 57 400 to about 71 400 sec-ft.

It is self-evident that under intelligent management the lake outflow would not be kept uniform throughout any series of years nor even throughout any single year. The aim would be to deliver the stored water into Lakes Huron and Michigan in the year and at the season of the year when it would do the most good.

In view of the fact that the natural channel of St. Mary's River will always be supplemented by power canals of large capacity and by the navigation canals on both sides of the river, it seems reasonable to anticipate that the ultimate total capacity of the lake outlets may reach 150 000 cu. ft. per sec. at a mean lake stage. Outflow at this rate is equivalent in one month to a layer of water 0.44 ft. deep on the surface of Lake Superior. In such a year as 1869, in which the lake rose 2.7 ft. in the 6 months from March to September, the lake elevation under complete regulation and this assumed capacity of outlet channels could be held down a little more than 1 ft. below the elevation then reached.

On the other hand, a delivery of water in the amount named from Lake Superior into Lakes Huron and Michigan is equivalent to a layer of water, over the whole surface of these lakes, of 0.30 ft. for each month of such flow. When it is recalled that there has been a year in which St. Mary's River discharged only 57 400 sec-ft., and that

there has been a mean monthly discharge as low as 45 000 sec-ft., it will be readily understood that an assured mean annual delivery in excess of 65 700 sec-ft., and a possible delivery of about 150 000 sec-ft. for a part of the year, will have an appreciable effect on the water stage of Lakes Huron and Michigan. Such regulation will not, as a matter of course, change the mean reduction of water level that would result from the continuous withdrawal, during a long period of time, of a given quantity of water from these lakes; but it will, under intelligent management, result in distributing the amount of depression of the water surface to the several months of the year, and to successive years, so that the lowering of the lakes due to water withdrawal will be least in the months when the lakes are lowest. In other words, by regulating the inflow from Lake Superior, the effect of the withdrawal can be, at least partially, offset at low lake stages. This effect can thus be artificially made maximum when the lakes are high, when there is no injury therefrom to navigation interests, and can be kept at a minimum when lowering would be detrimental to navigation. The effect of controlled water storage in Lake Superior, whether it be restricted to the 1.5-ft. or the 2.5-ft. limits, will be unquestionably of measurable benefit.

The desirability of utilizing Lake Superior as a storage reservoir, together with the utilization of the controlled flow of St. Mary's River to offset the effect of water diversion at Chicago for sanitary purposes, appears to have been pointed out first by Rudolph Hering, M. Am. Soc. C. E., in his report of October 15th, 1907, on the disposal of Calumet sewage.

While it is apparent that in such a year as 1892 the control of storage in Lake Superior would have made it possible to increase the mean annual flow of St. Mary's River from 57 400 to about 65 700 sec-ft., an increase of 8 300 sec-ft., or enough to raise the level of Lakes Huron and Michigan about 2 in. (2.4 in. less the effect of increased flow of St. Clair River due to greater lake elevation), it must be remembered that the effect of this storage upon the mean stages of Lakes Huron and Michigan for a series of years would be comparatively slight. This effect, notable though it may be for a single year or for several years in which stored water increases the flow of St. Mary's River, is offset in a long series of years by the fact that, in the years of more than normal water production, the water delivery into

the lower lakes must be cut down below natural flow, otherwise there would be no water for the improvement of conditions in those years in which the natural outflow from Lake Superior is small. The permanent effect of a controlled outflow from Lake Superior upon the stages of the lower lakes, averaged for a long series of years, would be due mainly to the slight modification of the outflow from these lakes resulting from the modified lake elevation. This effect would be slight, and may prove difficult to trace.

Whether an arrangement for storage in Lake Superior, with a range of annual mean elevations as great as 2.5 ft. to secure maximum benefit, can be effected, is not known at this time. It seems possible that this range might be attainable, in view of the fact that the extreme range of monthly means (1860-1905) has been 3.4 ft., and that, if the unusually high stage of 1838 be taken into account, this range has been about 4.5 ft. A computation of the effect of the larger storage upon the flow of St. Mary's River has been made, as above set forth, to emphasize the point that controlled storage in Lake Superior can be made beneficial to lake navigation, and, therefore, would be a partial offset to any water diversion from Lakes Huron and Michigan.

Based on the water elevations for the lakes, and the amount of flow in the St. Mary's and the St. Clair Rivers, as noted in the reports of the U. S. Army Engineers, it is found that a continuous withdrawal of 10 000 sec-ft. would lower Lakes Huron and Michigan about as follows:

For a mean annual elevation of 580 ft., the lowering would be 0.49 ft.

For a mean annual elevation of 581 ft., the lowering would be 0.47 ft.

For a mean annual elevation of 582 ft., the lowering would be 0.45 ft.

For a mean annual elevation of 583 ft., the lowering would be 0.43 ft.

A continuous withdrawal of this amount of water during the period 1860 to 1905 would have reduced the water elevations by a mean amount of about 0.46 ft.

As already shown, the offset to this depression, due to storage in Lake Superior during a year, corresponding to a year of minimum

flow of St. Mary's River, such as 1892, would be about 0.17 ft. (storage taken at 1.5 ft.).

The stages of Lakes Huron and Michigan have been lower during the years since 1888 than during the years covered by records preceding that date. Not only has the mean for the whole series of years been lower by about 1 ft., but an extreme low mean annual stage was reached in 1896, which was about 1.35 ft. lower than any recorded low water (annual mean) during the earlier period 1860 to 1888. The year 1888 is a convenient line of division in making this comparison, because the lake stage in that year was about normal, and has since remained less than normal.

The mean lake elevation from 1860 to the close of 1888 was 581.89 ft. The mean lake elevation, 1889 to 1905, inclusive, was 580.45 ft. The lakes were, therefore, about 1.44 ft. lower in the later 17-year period than in the earlier 29-year period. The mean lake elevation determined for the 46 years, 1860-1905, is 581.36 ft. The mean stage of the lakes during the 17 years, 1889-1905, was, therefore, lower than the mean for the entire period by 0.91 ft.

The long-continued depressed stage of Lakes Huron and Michigan has been attributed by the Board of Engineers on Deep Waterways to the enlarged section of St. Clair River and the improvement of the outfall from Lake Huron into the river. On this subject, the Board says:*

"There is now a channel over 40 feet deep from the lake into the river, the increased outflow through which has lowered the general level of Lakes Huron and Michigan about 1 foot."

The Board of Engineers also says:†

"The mean level of Lake Huron is apparently about 1 foot lower than it was fifteen years ago, which change has resulted from the enlargement and deepening of channels for waterway improvements and from the natural erosion of the bed of the river at the outlet of the lake."

The general depth of the foot of Lake Huron, 1½ miles above the head of St. Clair River, is stated by the Board of Engineers to have been originally from 21 to 27 ft., with numerous shoals 16 to 18 ft. deep. A channel, 2 400 ft. wide and 21 ft. deep, at a mean stage, has

* "Report of the Board of Engineers on Deep Waterways between the Great Lakes and the Atlantic Tide Waters," 1900, p. 37.

† "Report of the Board of Engineers on Deep Waterways between the Great Lakes and the Atlantic Tide Waters," 1900, p. 83

been cut through these shoals. Concerning the deepening of water in the head of St. Clair River by the scouring action of the water, the Board of Engineers states that, in 1867, surveys showed the depth of water on the bar over which the lake discharges into the river to have been 27 ft., and the central depth of water in the gorge at the head of the river to have been 48 ft. The surveys of 1898 and 1899, according to the Board's report, showed that a channel had been scoured through the bar to a depth of 75 ft., and that the water depth in the gorge at its narrowest point was from 48 to 66 ft.

The 15 years, 1885 to 1899, inclusive, to which the Board of Engineers referred in making its comparison of lake elevation in recent years with former elevations, show a mean water-surface elevation which was 0.95 ft. lower than that of the 25 years, 1860 to 1884, inclusive.

The possibility that altered conditions at and near the head of the St. Clair River had the effect of modifying, to some extent, the stage of Lakes Huron and Michigan must be admitted, but changes of outlet capacity have probably been only minor factors in producing the low lake stages, if indeed they have been of any effect. This will appear from the following consideration:

It must be apparent that the lowering resulted either from the cause to which it is attributed by the Board of Engineers, or there must have been less water presented by Lake Huron for delivery through the St. Clair River. Both causes may have contributed to the result. If the depressed water surface can be accounted for in part by a decreased water production of the drainage basin which is tributary to St. Clair River, then only the remainder, if there be any, will be ascribed to the increased outflow capacity of the head of St. Clair River.

That there has been a deficient water production in the drainage basin treated as a unit of the three lakes, Superior, Huron, and Michigan, during a long period subsequent to 1888, is primarily indicated by a decrease of the St. Clair River discharge. This decrease is quite as noticeable as the depressed lake elevation, as will appear by inspection of the diagrams on Plate XII. The flow of the St. Clair River, however, does not by itself represent the water production of the lake drainage basin from year to year, because some of the water remains stored in the lakes. It will be instructive, therefore, to

determine the annual water quantities in excess of evaporation received by the two lakes, Huron and Michigan, from all sources, and to compare these quantities with each other for the same time periods for which comparisons of lake elevations have been made.

The net quantity of water thus annually received by the two lakes is found by adding to the annual outflow through the St. Clair River the storage accretion (plus or minus as the case may be) of the two lakes, and adding also the diversion through the Chicago Drainage Canal, which has been about 4 200 sec-ft. since January, 1900.

The result of this computation, based on the figures contained in the Wheeler and other U. S. Engineer reports, shows conclusively that there has been a very decided falling off in the water-yield of the Huron-Michigan drainage basin in recent years, and that the drop in the water-surface elevation of the lakes is coincident with this decrease of yield. The water-yield of the basin (run-off, and rain on the lakes, less evaporation), as computed, noted for periods of about five years and expressed in second-feet continuous flow, has already been noted in Table 6 and is shown on Plate XII.

For the 17 years, 1889 to 1905, the entire drainage basin of Lakes Huron and Michigan, including Lake Superior, contributed to the lakes a mean flow of 175 000 sec-ft. of water, whereas the normal water yield of the basin is about 195 200 sec-ft. The deficiency of 20 200 sec-ft. is enough, at the normal stage of the lakes, to account for about 0.94 ft. of deficient water elevation. This being the case, the lakes would have been, at a mean elevation, nearly 1 ft. lower than normal subsequent to 1888, even though outlet conditions from Lake Huron had not changed.

It has already been shown that the mean elevation of the two lakes for the period 1860-1905 was 581.36 ft. and for the period 1889-1905, 580.45 ft. Had there been no withdrawal of water from the lakes at Chicago, these elevations would have been 581.38 and 580.50, respectively. The lakes would have been 0.88 ft. lower, in the 17-year period following 1888, than their normal elevation. This depression of 0.88 ft. is the combined effect of less than normal rainfall (in part perhaps more than normal evaporation) and of changes at the head of the St. Clair River. As above set forth, however, the effect on lake elevation that might be reasonably attributed to the first cause alone, viz., climatic conditions, is about 0.94 ft., slightly in excess of

the actual depression, leaving no drawing-down effect to be ascribed to the changes at the head of St. Clair River.

The conclusion that the channel changes at the head of the St. Clair River have not materially affected the stage of the lakes, if at all, is, as already stated, based on the lake elevation and discharge figures published by the United States Army Engineers. It is noteworthy that these figures show that the material reduction, after 1888, in the water productiveness of the drainage basin of Lakes Huron and Michigan, was confined almost entirely to the area directly tributary to these two lakes. There was no pronounced falling off noted in the discharge reaching these lakes through St. Mary's River from Lake Superior. However, as the published stream measurements and river stages for the same period of 17 years show a decrease in the discharge of St. Lawrence River, the outlet of Lake Ontario, amounting (after correction for depletion of lake storage and the diversion at Chicago) to about 18 000 sec-ft.—nearly as much as the decreased water production of the area tributary to the St. Clair River, 20 200 sec-ft.—there would appear little room for doubting the substantial accuracy of the published discharge tables of the St. Clair River, and, therefore, the conclusion, as stated, relating to the main cause of the low lake stages, seems to be based on reliable premises.

That this conclusion, which attributes the low lake stages of Lakes Huron and Michigan to a long period of deficient water production in the Huron-Michigan basin, and in a very small degree, if at all, to changes at the head of the outlet channel, is probably correct, is borne out by a statement of E. E. Haskell, M. Am. Soc. C. E., in his report of July 16th, 1900,* to the effect that what had a year previously been reported as a clear case of enlargement by scour of the head of St. Clair River was based on a comparison of a preliminary survey of 1898, with a chart based on a survey in 1867, and showed an 18-ft. cut over a portion of the gorged reach. Mr. Haskell goes on to say that an older chart of 1859 was subsequently found to agree much better with the survey of 1898. He caused old notes to be replatted, and careful comparisons based thereon have led him to the conclusion that the changes have been small. Between 1859 and 1867, the most restricted section may have been enlarged 9 000 sq. ft.

* Report of Chief of Engineers, U. S. A., 1900, Pt. 8, p. 5323.

Between 1867 and 1900 the changes, if any, Mr. Haskell says, are unimportant.

On the assumption that the discharge of St. Clair River has been approximated with a fair degree of accuracy for each year of the entire period, 1860 to 1905, it appears reasonably certain that the low stage of Lakes Huron and Michigan will not persist, that normal weather conditions will restore about 1 ft. of the lost mean lake elevation, and that the only depression below the original normal will be a small amount, if any, due to the St. Clair River changes and a small amount due to the Chicago diversion. Until the Chicago diversion is increased above the present amount of about 4 200 sec-ft., a mean lake stage at about 581.20 ft. is to be expected. This, it may be stated, is 1.70 ft. higher than the lowest mean annual lake elevation (579.50 ft.) of the past 48 years, which occurred in 1896.

This conclusion relating to a probable future higher lake level than that of the period subsequent to 1888 is inevitable, because it may be accepted as a certainty that the unusual climatic conditions which, since 1888, have resulted in a deficient output of water from the Huron-Michigan basin will not continue indefinitely. The occurrence of another period, as protracted as the one subsequent to 1888, of small water-yield in the lake basin is highly improbable. It is proper, therefore, to assume that the years in which such low levels as those of 1895 and 1896 will occur will be few and far between.

What has occurred in the past, however, may occur again; therefore means should be sought to keep up the elevation of Lakes Huron and Michigan during any future periods of deficient precipitation in the lake basin.

To some extent the raising of the level of these lakes can be accomplished with controlling works in the head of Niagara River. Such works have been recommended, and designs therefor have been made by the Board of Engineers on Deep Waterways. This Board proposes a submerged weir 2 900 ft. long and a series of sluice-gates, 13 in number, and each 80 ft. wide. These works, it is stated, would raise the level of Lake Erie 3 ft., the level of Lake St. Clair about 2 ft., and the level of Lakes Huron and Michigan about 1 ft. No attempt has been made to check this forecast. It is possible, moreover, to go a step farther and to provide works for the throttling of the waterway, or for the complete control of flow, within fixed limits, of

the St. Clair River. The stage of Lakes Huron and Michigan could then be brought under adequate control, and the lake stage could be maintained at all times as high as required by navigation interests.

That regulating works will sooner or later be constructed, in the rivers draining the several lakes, of such a character that the elevation of the lakes under proper management cannot fall below predetermined minima, is reasonably certain. It follows, therefore, in view of the benefit that will be derived from the use of Lake Superior as a storage basin, and of the control of water surface that will result from works below Lake Huron, that navigation interests will be but temporarily, if at all, affected by a diversion of water at Chicago, or elsewhere within reasonable limits as to amount, and that any deleterious effect will not continue beyond the time when the water elevation of this lake and Lake Michigan will be controlled by works below Lake Huron.

The stages of Lake Ontario and of the St. Lawrence River are questions apart from the one that has here been discussed. Enough has been said, however, to show that here, too, the less than normal stages of recent years are to be ascribed to climatic conditions, supplemented in a very slight degree only by the diversion at Chicago. The ultimate effect of the diversion of water from any of the lakes, offset by the equalizing effect of lake regulation, upon the stage of water in the St. Lawrence River is a study that will have to be made when the regulating works come under consideration.

DISCUSSION.

Mr. Chittenden.

H. M. Chittenden, M. Am. Soc. C. E. (by letter).—As a result of his investigation of reservoir possibilities in the arid regions in 1897, the writer became deeply interested in the subject of reservoirs in general as regulators of stream flow, and particularly in the great natural system of the St. Lawrence Basin. With the assistance of James A. Seddon, M. Am. Soc. C. E., a mathematician of exceptional ability, he undertook a study of the general problem of the interrelation of reservoirs in a descending series like those of the Great Lakes, where each unit, except the upper one, receives an independent supply from its own water-shed and a transmitted supply from the reservoirs above.*

This study, the first of its kind ever made, led to the formulation of certain principles controlling the action of such reservoirs upon their outlets and upon each other, and also to the enunciation of certain conditions and limitations which must govern in any scheme for the control of the reservoir levels. The Board of Engineers appointed under the Act of Congress of June 4th, 1897, to investigate and report on a deep-water route from the Lakes to the Seaboard was then just beginning its work, and the late George Y. Wisner, M. Am. Soc. C. E., a member of the Board, criticized quite severely some of the writer's conclusions, but apparently coincided with them later, in the main, as shown in his own report to the Board.†

At the time of the writer's studies the question of lake level regulation was very prominently before the public. The low stage to which some of the lakes had fallen, due to a series of unusually dry years, had occasioned a good deal of anxiety among commercial interests on the lakes, and many propositions were put forth as to possible correctives of the abnormal conditions then existing, and even for the improvement of normal conditions. It was claimed that the fluctuation of the lake levels could be limited to a range of 6 in. or less, that the levels of all the lakes could be permanently raised, and that Lake Superior could be utilized as a storage reservoir to help maintain the levels of the lakes below, and particularly to compensate for the prospective loss through the Chicago Drainage Canal then nearing completion. A scheme for the control of Lake Erie, frequently proposed, was a long-crested weir around the head of Niagara River, the development of the crest being sufficient to regulate the discharge automatically so that the surface of the lake would never rise or fall beyond certain prescribed limits. It was also held by some that the regulation of Lake Erie would react through the channels above it and regulate the levels of Lake Michigan-Huron in like manner.

* *Transactions*, Am. Soc. C. E., Vol. XL, p. 355.

† Report of the Board, p. 274.

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Among the conclusions arrived at in the writer's paper were the following:

That the regulation of the fluctuation of the levels of the upper lakes to a limit of 6 in. was a physical impossibility on account of the great volume of water lost during the summer through evaporation;

That any material restriction of the annual fluctuations of the lake surface was of doubtful practicability, because such restriction could only be had at the expense of uniformity of flow in the outlets, and this would probably be inadmissible from considerations of navigation;

That the periodic, or cyclic, fluctuation of level, extending over a series of years, could be eliminated altogether and the lakes could be kept from falling below a definite level that might be established;

That the controlling works in the Niagara could not be in the form of a fixed weir, automatic in its action, but must be in the form of a movable weir by which the flow through it would be under human control, and adjustable to the daily conditions of wind and supply of water to the lakes;

That control of Lake Erie would not suffice for the control of Lake Michigan-Huron;

That the compensating effect of storage in Lake Superior for any permanent diversion through the Chicago Drainage Canal, or for a permanent enlargement of the outlets, was visionary and impracticable.

As above stated, the first three of the foregoing conclusions were practically recognized in the report of the Deep Waterways Board. As to the fourth conclusion, the Board found that raising the level of Lake Erie 3 ft. would permanently raise Lake Michigan-Huron about 1 ft. In regard to the use of Lake Superior as a storage reservoir, the Board was silent, and was therefore presumably of the opinion that such use to compensate for permanent diversions from the lakes below was impracticable. The present International Waterways Commission recently made a special investigation of the effects of the Chicago Drainage Canal diversion upon the levels of the lakes and of the means of compensating for the diversion. Nothing is said in their report as to storage in Lake Superior, and they likewise apparently considered the scheme impracticable. Since the date of the above-mentioned reports, Rudolph Hering, M. Am. Soc. C. E., has come out in strong advocacy of this scheme,* and now Mr. Grunsky's very able paper is under consideration.

That two such competent authorities should advocate the plan has led the writer to review the reasoning by which his own conclusion was arrived at. He is still unable to see how storage in Lake Superior can compensate for a permanent diversion at Chicago or at any other point, or for a lowering due to an increase in the dimensions of any

* "Report on the Disposal of Sewage from Calumet Subdivision of the Sanitary District of Chicago," October 15th, 1907, pp. 28-31.

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of the outlets. The reason for this is that the mean levels of the lakes are dependent upon two things: the character of the outlets and the supply of water, using this last term in its algebraic sense to include the negative effects of evaporation and diversions. The outlets remaining the same, any permanent diminution of supply must result in a permanent lowering of mean level, and the only way to prevent this, without modifying the outlets, would be to compensate for this loss of supply. Can storage in Lake Superior accomplish this purpose? Manifestly not, because the total supply upon which the mean level depends cannot be affected by any manipulation which can be made of a portion of that supply, so long as its quantity remains unchanged. Storage in Lake Superior cannot affect the total supply to Lake Michigan-Huron in the least. Any increase in storage above the normal can be accomplished only by restricting the outlet discharge below the normal, and this must result in a proportionate lowering of the levels of the lakes below while the accumulation is going on. When the storage is run out, it can do no more than make up for the loss which its previous withdrawal had occasioned. Of course, if the storage is accumulated very slowly through a long period of time and then run out very rapidly through a short period, a greater increase of gauge height will result than the previous decrease; but, if the durations of the diminished and increased stages are considered, the account will balance. Whatever may be done with Lake Superior storage, therefore, the supply to Michigan-Huron cannot be increased; and the diversion at Chicago thus represents a permanent and uncompensated loss.

The storage in Lake Superior can be manipulated so as to reduce fluctuations of level in Lake Michigan-Huron, but only at the cost of a greater increase in the fluctuations of the upper lake. How far the advantage in one case may be offset by the disadvantage in the other should control in determining the extent to which such regulation should be applied; but the reduction of annual fluctuations of level in Lake Michigan-Huron is a very different thing from compensating for a permanent loss of supply so as to maintain the mean level of the lakes. Manipulating the storage of Lake Superior cannot affect in the least the supply to the lakes, and is therefore powerless to affect their mean level or compensate for diversions. In fact, the writer does not understand that Mr. Grunsky really claims this, though his language leaves some doubt in the writer's mind as to his exact meaning.

Clearly, a permanent diversion from the lakes, at any point, of a given quantity of water or an enlargement of the navigable channels by artificial means can be compensated for only by restricting the flow through the outlets below. Theoretically, this is perfectly feasible; practically, it is difficult of accomplishment owing to the conditions

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imposed by navigation. In Lake Superior the difficulty is much less because commerce has to pass through a lock anyway, and controlling works in the river would not interfere with navigation; but in all the outlets below the case is different. Controlling works which should diminish the normal discharge would tend to concentrate the slope at the site of the works, increasing the velocity at such points, while the diminished flow might leave deficient depths in the channels below. These drawbacks to navigation might prove too great to be readily overcome. To construct works of such character and magnitude as to maintain the necessary slopes and depths in these channels during periods of restricted flow would assuredly be very costly.

There is, however, a considerable period of each year when navigation is suspended, and there is no obvious reason why, during this period, the flow of the outlets may not be cut down to whatever point would be necessary to raise the levels of the lakes to the desired heights before the next navigation season opens; but if the regulated mean level were elevated above the normal mean level, the increased outflow during the navigation season, in the absence of works regulating the slope and permitting a partial closure of the channels at such times, would probably increase the subsidence of levels during such periods as compared with that under normal conditions.

That the mean levels of all the lakes can be permanently raised; that the periodic or cyclic fluctuations can be eliminated; that some reduction can be made in the annual fluctuations; that a much greater diversion at Chicago than 10 000 cu. ft. per sec. can be compensated for, by restricting the flow through the outlets, are measures which the writer believes to be within the resources of river engineering. As in any other project, the real question is one of cost in relation to the benefits to be received.

Mr. Grunsky.

C. E. GRUNSKY, M. AM. SOC. C. E. (by letter).—It is stated in the paper that the conclusion reached with reference to the small effect upon the stages of Lakes Huron and Michigan, that has hitherto resulted from channel enlargement at the outlet of Lake Huron, is based on the assumption that the discharge from these lakes, as far back as 1860, has been determined with a fair degree of accuracy by the United States Engineers.

The rating tables, based on measurements made in recent years, and the observed lake stages, are the basis of the discharge tables. If any material channel enlargement, natural or artificial, preceded the gaugings on which the rating tables are based, then the application of these tables to water stages under original conditions would give too large results. Can the higher stages of Lakes Huron and Michigan preceding 1888 have been possible with less outflow than indicated by the record? This question has already been answered by reference to the fact that the interpretation of the stages of the St. Lawrence River shows

Mr. Grunsky. substantially the same decrease of flow in recent years as the St. Clair River. It remains to be added that, if too great a flow has been recorded for the earlier period, the error would be a progressively decreasing one from 1860 to the time of the gauging. The increasing water yield for successive 5-year periods from 1860 to 1885, as shown in the last two columns of Table 6, negatives the probability of any such error.

The statement is clearly made in the paper that the control of the outflow from Lake Superior would not change the mean reduction of lake level that would result from the withdrawal, during a long period of time, of water from the lakes. In other words, the controlled outflow from Lake Superior will not appreciably modify the normal stage of the lower lakes; but it is claimed that such control would be beneficial to navigation interests, because thereby the lakes would be prevented from dropping as low as they would otherwise go.

Colonel Chittenden has clearly pointed out some of the difficulties that must be overcome in putting into practical effect a regulation of the outflow from the lower lakes; but the regulation can be effected. It is, at any rate, as pointed out in the paper and as restated by Colonel Chittenden, theoretically feasible to modify the lake stages beneficially by controlling the outflow, and to this fact attention must be directed when the question of setting a limit upon the amount of water that may be diverted from the lakes is under consideration.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1101

THE FLOODS OF THE MISSISSIPPI DELTA: THEIR CAUSES, AND SUGGESTIONS AS TO THEIR CONTROL.*

By William D. Pickett, M. Am. Soc. C. E.*

One of the great engineering problems of the age, if not the greatest, is to obtain such control of the waters of the Mississippi River as to prevent those periodical overflows which, in the past, have devastated and made desolate the entire area of its delta. This delta is probably the most extensive in the world, and has been formed, during past ages, by the discharge of the waters of the greatest river in the world, certainly the longest, in miles, and with a volume of discharge not exceeded by any, unless it be the Amazon, in South America.

From a geological standpoint, the most plausible theory in regard to the formation of this delta is that at one time the Gulf of Mexico extended as far north as the confluençe of the Mississippi and Ohio Rivers; that, in past ages, by the discharge of the silt brought down in the annual floods, the land has been gradually formed and extended out into this inland sea, until, about the beginning of the nineteenth century, it had assumed the form and area existing to-day.

When it is considered that the level of the low-water stage at Cairo is about 250 ft. higher than the mean tide level of the Gulf, the

*As a matter of interest, the fact is here mentioned that Mr. Pickett has been a Member of this Society since July 6th, 1853, having joined it during the first year of its existence. —*Secretary*.

centuries of time required to develop this delta and the depth of its soil of almost illimitable fertility may be realized.

Following a natural law, the land on the banks of this river is higher than that farther inland. The same conditions obtain on Deer Creek and Sunflower River, in the Yazoo Basin, as on the Bayou Plaquemine and other large bayous draining the lower portion of this delta. A cross-section of the Yazoo Delta, near the latitude of Greenville, Miss., made by the Engineer of the Yazoo Levee System previous to 1861, developed the fact that the level of the bank of the main river was about 13 ft. higher than comparative levels 20 or 25 miles inland near Deer Creek and Sunflower River; such was the writer's information at the time.

As a natural sequence to these conditions, the "high-lying lands," on the main stream and such subsidiary streams as those noted above, were first brought under cultivation, and, being of unexcelled fertility, produced the heaviest yield of cotton, corn, and other crops suitable to the climate.

The area of these "high-lying," and, of necessity, more valuable lands, is a small percentage of the area of this delta. A much larger area consists of "low-lying lands," designated by the U. S. Geological Survey as "Sharkey clay," which are more or less subject to annual overflows, and are covered with a heavy growth of timber, such as white oak, red oak, water oak, cypress, and gum, all more or less valuable for commercial purposes.

The cost of clearing off this heavy growth of timber and preparing the ground, by ditching, etc., for cultivation, together with the almost certainty of periodical overflows, has heretofore and will hereafter prevent these otherwise valuable lands from being brought under cultivation and rendered productive until the levees on the main river have been perfected so as to render these overflows improbable, if not impossible.

These low-lying "overflow lands" are represented, by officers of the Geological Survey and by intelligent planters, to be of as great fertility and as productive of corn and cotton as the "high-lying" lands on the streams, where the ground has been properly prepared. With land of this character, protected from overflow, it is believed that the value of its timber for commercial purposes will go far toward meeting the cost of preparing the ground for crops, if it does not equal or surpass it.

These facts being substantially true, it is proposed to investigate and ascertain approximately the value, to the nation, of these "overflow lands" on the supposition that they could be freed from that incubus of "overflow" so much dreaded by the Mississippi River planter.

The most reliable information as to the extent of this entire delta is obtained from the Agricultural Department, through Mr. J. A. Bonsteel, of the Bureau of Soils, who, from the most reliable records accessible, estimates this area at 30 526 sq. miles. From the surveys of that Bureau in the Yazoo Delta, it is ascertained that 65% of the total area is "overflow" land, leaving 35% for land under cultivation. Adopting this basis for the entire delta, it indicates the total area to be, in round numbers, 30 000 sq. miles, and the area of "overflow" lands to be 20 000 sq. miles of land of the highest fertility, yet it is of no value, and never will be, for crops, for causes before described.

From the writer's knowledge of the conditions in this delta, it is believed that the land under cultivation is nearer 25% of the total area than 35 per cent. The 35%, however, has been adopted in the following estimate furnished by Mr. Bonsteel:

"Area of Mississippi Delta, in Square Miles, by States:

"State.	"Total area.	"Area of overflow land.
"Kentucky	150	100
"Tennessee	398	260
"Missouri	3 111	2 022
"Arkansas	5 406	3 513
"Mississippi	6 966	4 425
"Louisiana	14 495	9 425
	"30 526	19 745."

In making an estimate of the yield of this now non-productive land, the area of Kentucky, Missouri, and 100 sq. miles of Tennessee is assigned to corn; 160 sq. miles of Tennessee and all of Arkansas and Mississippi are assigned to cotton, and all of Louisiana is assigned to sugar and rice. As the necessary statistics for the yield of Louisiana lands have not been attainable, a yield of $35 per acre has been estimated, the same as the yield of cotton lands.

To compensate for lands not available for crops in the "over-

flow" area, such as that occupied by rivers, bayous, and creeks, 10% has been deducted. The results, in round numbers, are as follows:

Corn land............	2 000 sq. miles	= 1 280 000 acres.
Cotton land..........	7 300 " "	= 4 672 000 "
Sugar and rice lands...	8 500 " "	= 5 440 000 "

In placing a value on crops from these lands, the yield of corn has been taken at 35 bushels per acre, and the value at 50 cents per bushel. The yield of cotton has been taken at $\frac{7}{8}$ bale per acre and the value at \$40 per bale. The yield of sugar lands has been taken at \$35 per acre, the same as cotton lands.

Turning these statistics into dollars and cents, we have an estimate —conservative, it is thought—of the value of crops that can be realized from land that never has had and never will have much intrinsic value for agriculture until freed from that overhanging incubus—the periodical overflows from that mighty river. The little benefit that has been realized in the past, from this overflow land as a range for stock, has been much more than counterbalanced by the destruction of stock in such overflows.

1 280 000 acres of corn land at 35 bushels per acre = 44 800 000 bushels at 50 cents..............	\$22 400 000
4 672 000 acres of cotton land at $\frac{7}{8}$ bale per acre = 4 088 000 bales at \$40 per bale.............	163 520 000
5 440 000 acres of sugar land at \$35 per acre............	190 400 000
Total value of crops........................	\$376 320 000

Bear in mind that this estimate is based on the yield of fresh land, which should be much greater than that from the old plantations of this delta which have been under continuous cultivation for 50 or 60 years, or more. Bear in mind, also, that it is based on what the ground will bring forth with average cultivation, not on the profit to the planter. Bear in mind, also, that this estimate is exclusive of the 35% (9 000 sq. miles) of this delta which is supposed to be under cultivation and already producing crops. In years of widespread overflows on this delta, the crops on this area of 9 000 sq. miles will be a total loss to the nation, increasing the above estimate of \$376 320 000 by the loss of the crops on that 5 700 000 acres of cultivated land.

The annual loss to the United States appears to be $370 000 000 in a small area of its territory, for want of protection against the overflow of this river.

As an indication that the writer's views of the value to the nation of this small fraction of its area are within conservative lines, an extract is given from the conclusion of an address before the Commercial Club of Cincinnati, in December, 1903, by B. M. Harrod, Past-President, Am. Soc. C. E., one of the oldest and ablest members of the Mississippi River Commission, bearing directly on this subject:

"Now, this is the proposition: There is an area of 20 000 000 acres of the most fertile land in the world. At least three-quarters of it are susceptible of the highest cultivation. Its potential products are diversified; including wheat, corn, cotton, sugar and rice. The timber wealth of the valley is immense, and it is in every way favorable from climate, soil and means of transportation for development by the farmer, the manufacturer, and for railroads. Without levees it is a jungle; an uninhabitable swamp. With levees each and every acre can be protected at a cost not exceeding three dollars, and this protection can be maintained at an annual cost of ten cents per acre. Nearly two-thirds of the work is now done."

The prospective addition of 4 000 000 bales of cotton and a proportionate yield of sugar—both among the necessities of life—to the resources of the country, renders the subject one of National importance. It seems to be a matter of paramount importance, and demands an investigation as to the causes of this condition of things, and to ascertain if means cannot be suggested that will, at least, mitigate the effects of these periodical disasters.

The magnitude of this problem should suggest diffidence in approaching its solution. In the past, it has engaged some of the best and most experienced engineers of the country; whereas they have worked along proper and conservative lines, the means at their command have been entirely inadequate for carrying out their plans. At the beginning of 1861, the levee systems below Memphis were in good condition, as far as constructed. The ravages of war, however, and the unrestrained power of the river, exerted in some very high overflows, swept off almost the last vestige of the levees existing at that time.

Since the close of the conflict (1865) the States bordering on this river have made remarkable progress in the rebuilding of the levee system, until at the present time, there is substantially a continuous

system of levees on each bank from Cape Girardeau to a point below New Orleans. Heretofore, each State has controlled its own levee system, but with means entirely inadequate to the magnitude of the work. It is believed there should be one controlling head for the entire levee system of the delta, and there should be ample means to carry out its plans. It is believed, moreover, that in a few years Congress will recognize the national importance of this work, in connection with the plans for the deep-water navigation of this great inland sea, and provide ample means for carrying out the plans determined on by this central control.

The writer has had especial opportunities for the study of the conditions at both ends, as he conceives it, of this great problem, and desires to record, in a tentative way, the views and opinions that have impressed themselves upon him. From 1856 to 1861, as a civil engineer, he had charge of the construction of the Memphis and Ohio, one of the principal railroads leading from Memphis, Tenn. Subsequently, 1867 to 1873, after the ravages incident to the war, he had charge of the reconstruction of the same road. During that period he witnessed a great many of the floods which devastated the delta. In 1866 he was a cotton planter in the Yazoo Basin, and was a sufferer from the flood of that year. During all this time, with the predilections of a civil engineer, he naturally studied all questions bearing on the causes of these floods. He sought information from all accessible sources; from old and experienced river men—the captains and pilots of steamboats—from cotton planters, and from civil engineers.

During these periods of overflow, this delta became an inland sea, 40 miles wide opposite Memphis and from 70 to 80 miles wide opposite the Yazoo Basin, with an occasional island where spots of dry land on the plantations were in evidence. In the river channel, its waters were extremely muddy—angry-looking, awe-inspiring and repulsive. This indicates the extent of the writer's opportunities for forming correct opinions at the lower end of this problem.

It happened that in 1876 circumstances drifted him to the Northwest, where 28 years of his life were spent in Montana and Wyoming. Seven or eight months of each year for the first eight years were spent in the mountains at the heads of the various tributaries of the Missouri and Columbia Rivers, whence come the melted snows that

make up principally the "June rise" that occasionally has such a potent influence in the devastation of the Great Delta.

Twenty years were afterward spent on a cattle ranch on the Upper Grey Bull River, at an elevation of nearly 7 000 ft. above tide. Within ten miles to the south is Franc's Peak, the elevation of which is given by the Geological Survey as 13 300 ft. above sea. On each side of the valley are mountain ranges with elevations of 10 000 and 11 000 ft.

During those twenty years' residence in this locality, a daily record was kept of temperature and precipitation, especially of snowfall. The necessity for irrigation called attention to all conditions influencing the water supply. The minimum temperature during the winter months varied from 24° to 47½° below zero. The annual snowfall varied from 50 to 110 in., the latter being unusual and extreme. The average snowfall per year was from 75 to 80 in., the average annual precipitation being about 13 in.

The conditions obtaining in this locality, as regards temperature, snowfall, and snow melting, may be safely taken as those on all the tributaries of the Missouri River of the same altitude. This altitude is about the upper limit of even forage crops. Basing opinions and views on the observations and knowledge gained by a long residence on the Lower Mississippi, and on a longer residence at the upper end of this great problem, among the mountains at the headwaters of the various tributaries of the Missouri, the following condensed statement gives the results of this study, premising that it applies more especially to conditions as to the delta preceding the year 1861. During the "War between the States," the levees built previous to that date were destroyed to a greater or less extent, by military acts or by destructive floods.

Each year there were two periods of flood-waters in the Mississippi River below the mouth of the Ohio. The first flood, styled the "spring rise," came almost entirely from the Ohio River, reinforced to some extent by the melted snows and rains from the Lower Missouri and the Upper Mississippi proper. The advent of this flood at its mouth varied in time, volume, and intensity with the climatic conditions of each spring.

The second flood, styled the "June rise," came mostly from the Missouri River, caused, as a governing factor, by the melting of the

accumulated snows of winter at the sources of its larger tributaries having their origin in the great Continental Divide.

The advent of this flood at Cairo, and its volume and intensity, were dependent mostly on conditions at its head; the amount of snowfall during the previous winter on its eastern water-shed, more especially in the vast pine forests; the time, whether early or late, of the first warm spells of spring, their duration and intensity (it required a week or often ten days of warmth sufficient to melt snow at night, at an elevation of 8 000 ft., to produce the highest floods in the local streams); and, to some extent, on the local rains in the lower water-shed of the Missouri and Upper Mississippi Rivers. The spring floods from the Arkansas and Red Rivers, having their origin in the same range of mountains, but much farther south, did not contribute so seriously to the disastrous effects in the lower delta, because they usually came earlier, and had passed down before the advent of the "June rise" from the Missouri.

In a majority of years the crest of the "spring rise" had passed Cairo before the advent of the "June rise" from the Missouri, in which event it passed down to the Gulf with but little damage to plantations or levees. In cases where cultivated fields were overflowed, the water subsided in time to plant cotton and corn and to raise a partial crop of each.

Following in the wake of the "spring rise" came the "June rise," but, as the former had substantially passed out of reach, the latter passed down to the Gulf without doing material damage to either levees or plantations.

As is evident from this statement of conditions, if, from various climatic causes, the "June rise" from the Missouri came tumbling down upon the "spring rise" before the latter had passed out of the way, there would be one of those disastrous overflows which the system of levees built previous to that date had never withstood, resulting in enormous losses to the plantations, usually, in the entire delta. Such floods were so late in subsiding from the cultivated fields—sometimes as late as August 1st—as to prevent the raising of either cotton or corn for that season. The best information was that these disastrous floods came in cycles of about seven years, and varied in intensity, in accordance with the volume of water brought down in each of the two "rises," and, in a large measure, as to the point at or near Cairo, at which the two floods merged.

Since the writer's personal knowledge of conditions in this delta up to 1873, more than thirty years ago, these conditions have been modified to a certain extent. The "spring rise" from the Ohio River water-shed makes its advent below Cairo earlier and in larger volume than in former years, in a volume so much larger that the levees are often crevassed with the sequence of great damage to the planting interests. Although this "spring rise" may have passed down and out of the way of the "June rise," yet, if the latter was of an average, or greater than the average, volume, its waters would overflow through the "crevasses" made by the earlier rise, widening them and resulting in about the same amount of damage as would have occurred had the conditions been the same as they were previous to 1861.

Notwithstanding the somewhat earlier advent of the "spring rise" at Cairo, it has not prevented, at times, that conjunction of events which occurred in former years, when the two "rises" combined their floods there, with such disastrous effect in the delta below. The only perceptible difference is that these combinations of the two "rises" occur at somewhat longer intervals than seven years, as the writer is informed.

The much earlier advent of the "spring rise" at Cairo can be attributed to but one cause, namely, the denuding of its entire water-shed of the timber and underbrush which covered it thirty or forty years ago. The result, as was to be expected, is that the melted snows and rains are precipitated into its valleys much earlier and in larger volume, resulting in great destruction of property, and in hardships to the people of Pittsburg and the entire Ohio Valley below.

The destructive effects of the "spring rise" on the delta below are caused when the early floods from the Cumberland and Tennessee Rivers merge near its mouth with the floods from the upper tributaries of the Ohio.

This condition of affairs in the Ohio Valley is an object lesson as to that which will occur on a much larger scale in the great delta, unless we not only preserve, but add to, the extensive forests now existing at the sources of the principal tributaries of the Missouri, which furnish such a large percentage of the annual floods poured into it at Cairo.

Imagine for a moment that the forests on the eastern slope of the great Continental Divide could be suddenly wiped from the face of the mountain, and that their winter snows were exposed to the direct

rays of the sun in spring and summer; there would be such an early melting of the snows in the spring that the "June rise" from the Missouri would make its advent near Cairo so early every year that it would merge with the "spring rise" from the Ohio, overtop the levee system, and create such devastation that in a few seasons it would depopulate the entire delta, and New Orleans would become a thing of the past. This is not a fancy sketch. The same causes, however unrestrained, that have produced the conditions before alluded to on the Ohio River water-shed (the greed of Man), combined with the ravages of forest fires, which, when under full headway are as uncontrollable as the floods of the Mississippi, render this catastrophe not impossible. Fortunately for the country, this is not a supposable case at this time.

President Roosevelt, soon after assuming office, aroused public opinion as to the value of these forests for irrigation, so that Congress quickly responded in liberal appropriations, the Forestry Service was at once organized along liberal lines and under the direction of Gifford Pinchot, Assoc. Am. Soc. C. E., and everything points to a thorough oversight of this most valuable asset.

A brief study of the foregoing facts and views seems to point to a simple and evident solution of the problem set forth in the beginning of this paper, that is, to control the "spring rise" from the Ohio or the "June rise" from the Missouri so as to prevent their conjunction or merging near the mouth of the Ohio, which has always produced great destruction below.

Could this desirable object be effected, a system of levees in the delta adequate to take care of the "spring rise" from the Ohio would substantially be sufficient to take care of the "June rise" which, though of greater volume, would flatten out when not encountering any back flow from the Ohio flood.

There does not appear to be much relief in sight in reference to the control of the "spring rise" from the Ohio. Reforesting its water-shed would give relief to its immediate valley, but none below its mouth. On the contrary, the earlier advent of that flood at its mouth gives it the more time to get out of the way of the "June rise." The proposal to store the flood-waters of this stream in immense reservoirs, by impounding its waters behind dams, does not impress the writer as either feasible or practicable. The cross-section of its valley below

Pittsburg is too flat for reservoirs of large capacity, and, above that point, the valleys of its tributaries are too narrow and their gradients too steep to be available for that purpose. The same conditions exist as to its other important tributaries, the Cumberland and Tennessee Rivers.

For the past ten years the General Government has been engaged in building an extensive system of reservoirs for the purpose of impounding the spring and summer floods at the head of the Mississippi River, with the object of turning them loose in the fall to assist in the navigation of this river below St. Paul. While these works answer and will continue to answer the purpose of their construction, the volume of water impounded will not give material relief at Cairo.

It is, then, to the Missouri River that we must look for some means of holding back or retarding the advent of the "June rise," so as to give the "spring rise" sufficient time to get out of its way.

The building of artificial dams to impound these floods seems to be neither feasible nor practicable, within reasonable cost, as was contended regarding the Ohio River floods. The writer recalls sites for such dams on most of the large tributaries of the Missouri (the upper ends of cañons, with the valley of the stream above opening out into a comparatively broad valley), where, by building dams 500 or 600 ft. high, much of the flood-water of each stream could be held back. Without going into the calculations of the cost of such works on all the large tributaries, it would appear to be so enormous, when compared with the cost of the means suggested by Nature, and spread out all around these proposed dam sites, that comparison of the merits of each scheme would seem to be unnecessary.

On the head-waters of its principal streams, Nature has pointed out the means to be used in accomplishing this desirable end by providing those dense pine forests that serve as vast storage reservoirs for the accumulated deep snows of winter. These forests, by their shade, protect the snows from the direct rays of the sun, allowing them to melt so gradually as to prevent damage in the streams below. It is the rapid melting of the snow on the large areas on and contiguous to the Continental Divide, which from various causes have been denuded of their forests, that causes flood damage in the streams below, and is the main factor of the "June rise."

The lead pointed out by Nature must be followed. The present

area of these forests must not only be preserved, but must be added to, by reforesting the large areas of open or prairie land which are surrounded by or are contiguous to the main bodies of forested land.

On each slope of the Continental Divide, in Montana and Wyoming, and on the mountain spurs branching out therefrom, are vast pine forests extending from 30 to 60 miles in width on each side of the summit. There are also groups of mountains of considerable extent arising from the plains on each side and within 100 miles of this Divide (such as the Big Horn Range and the Teton groups), which are covered more or less with similar forests.

The limit of the timber line in that latitude is about 9 200 ft. above sea level. Below that line and above 8 000 ft. elevation, and surrounded by or contiguous to these forests, are open or prairie lands about equal in area to that of the pine forests, which have been denuded of timber, by fire and other causes, the conditions of soil and moisture being similar to those of the dense forests adjoining them.

On these upper mountain plateaus are to be found many never-failing springs, and these, during the dry months of the fall, will serve to irrigate the young pine shoots used in reforesting. The adjoining timber will furnish all the small pine saplings needed.

Above the timber line and on each side of the Continental Divide and the mountain spurs branching out therefrom are large areas of open land, but whether it has been denuded of its timber by fires, or whether its condition is normal and due to the frigidity of the climate in winter, is not apparent. On the eastern slopes there is generally a sufficient depth of good soil which has been drifted from the western slope by the high westerly winds prevailing at all times.

During the storms of winter, there form on the eastern slope snow-drifts deep enough to last very often until snow flies again. The writer is not sufficiently informed as to the principles of forestry to give an opinion as to whether these open areas above the timber line can be reforested; but there are large areas with sufficient soil, and water from the snow banks, for that purpose, if the winter temperature does not preclude the idea.

There are no records in existence as to the present extent of the pine forests on the head-waters of the Missouri River tributaries, nor of the open areas now denuded of forests from unknown causes, lying among or contiguous to the existing forests below the timber line.

It is believed, however (and this belief is based on a somewhat intimate knowledge of about 100 miles length of this Continental Divide and its numerous spurs), that below the timber line there are areas of open land which can be reforested, and that these are sufficient to equal in extent the areas of existing forests.

As before stated, the record of snowfall at the Four Bear Cattle Ranch, on the Grey Bull River, for a term of years, averaged about 80 in., and its climate could be taken as the average of conditions at points of similar elevation (7 000 ft.) around the water-shed of the Missouri River. It is thought that the snowfall on the high mountain plateaus is greater than in the valleys below, but there are no data at hand to sustain this opinion.

The snowfall, as measured each day, was of light weight, the 80 in. of average snowfall representing about 8 in. of water.

This 80-in. depth of snow has fallen between September 1st and May 1st, and, in the pine forests, at altitudes of from 7 500 to 9 200 ft., and represents all the rain and snowfall between these dates. At that altitude freezing temperature is one of the principal preservatives of snow, especially in the warm months. There are few nights at that season, among the pine forests, when snow does not "crust over" sufficiently to support the weight of a man or even a horse.

In estimating the amount of precipitation that goes to make up the floods of the Missouri River, there must be added to this 80 in. of snow the precipitation in May, June, and July of each year. This represents the total annual precipitation for the year, except that which occurs in August. As the record for August around this water-shed is only about $\frac{65}{100}$ in., it can be safely assumed that this annual precipitation of that altitude represents the stored up precipitation for each year in those pine forest reservoirs.

As will be hereafter explained, one-half of this precipitation (say, 6½ in.) represents the portion covering the open areas, surrounded by or contiguous to the forests, which turns into water in May or during the first days of June, and flows into the larger tributaries within 12 hours after melting, thus forming the controlling factor of the famous "June rise." The volume and intensity of this flood are caused, not only by the volume of water thrown into the stream within the space of a month, but by the melted snow being at once tumbled down the

steep mountain into the water channels below in an incredibly short space of time, thus adding to the intensity of the flood.

As contributory to the "June rise" may be mentioned the melting of the deep snowdrifts of the great plains west of the 100th meridian and the local rains of the Lower Missouri and Upper Mississippi.

Contributory to the "spring rise," to some extent, are the melted snows of the open areas of these mountains, that lie above timber line (9 200 ft.). As these snows lie usually in immense drifts, they do not melt much before the latter part of July and August, too late to affect the "June rise."

The volume and intensity of the "June rise" are dependent to a certain extent on the coincidence or conjunction of all these factors. The melted snows from the forest reservoirs are, undoubtedly, the controlling factor.

In April, sometimes earlier, under the influence of the "Chinook" or warm winds from the Pacific, the snows covering the plains below the foothills of the Continental Divide commence melting, causing the first floods of the main stream and tributaries, which soon completely sweep out the ice, often creating immense gorges down as low as Omaha.

The writer's information is that the influences of these Chinook winds at its head, frees the Missouri of ice much sooner than is the case with the Upper Mississippi.

By April 1st, the snows in the timber on the high mountain plateaus have usually settled to a depth of from 4 to 5 ft. On account of the low temperatures peculiar to an altitude of 8 000 or 9 000 ft., the snow does not usually commence melting in the open tracts until about May 1st. It melts so rapidly, however, that by June 10th the grass is in such an advanced stage that cattle from the ranches below are taken, at about that date, to these upper mountain plateaus to be kept there until the November snows.

In the forests contiguous to these open spaces, the snow does not usually disappear until the beginning of August.

This, in a nutshell, tells the value of forests in conserving the snows of winter. In other words, forests of average density and at the average altitude of their habitat, 8 000 to 9 000 ft., will preserve the snows of winter from melting two months longer than open areas exposed to the direct rays of the sun.

As an object lesson in support of this experience on the Grey Bull, the writer cites his experience on July 16th, 1880, in crossing the Continental Divide from the waters of Snake River opposite the Upper Geyser Basin of the Yellowstone National Park. The Divide is comparatively flat, has an elevation of about 8 200 ft., with about an average density of forests, and at that date was covered with a depth of from 3 to 6 ft. of snow, depending somewhat on its exposure to sunshine at the small openings in the timber. The area of these openings ranged from a few acres to 20 acres. There may have been some drifting of snow in places. As is always the case at this altitude, the snow banks are crusted over during the night from cold (one of the elements of snow preservation at this altitude), and the party was compelled to take advantage of it, and get the pack animals across the snow-drifts on the summit early in the morning. The small openings in the forests, before alluded to, had been freed from snow so long that grass was so far advanced in growth as to afford enough pickings for nooning for the horses. The previous winter had been an average one for snowfall, and that summer an average for heat. This experience afforded an admirable object lesson as to the efficacy of forests for preserving snow.

Now apply these facts and conditions to the entire Missouri River water-shed of like altitude, 7 000 ft. and above, to the Milk, the Sun, the Jefferson, the Madison, the Gallatin, the Yellowstone, the Big Horn and the Platte Rivers. As at present, one-half of the winter's snows that cover the open spaces will pass off in May or thereabouts, and will not be of benefit for local irrigation on the smaller streams, for it is not then needed, nor on the larger streams below for navigation, for local rains have already provided a sufficiency.

Now conceive the entire water-shed of this mighty river (below the timber line and well above the agricultural belt, say 7 700 ft.) to be clothed with forests of average density. When spring comes, the snow will commence melting in that belt about May 1st, and instead of half the winter's snowfall melting during that month, and causing as a main factor the "June rise," it will melt gradually, and will not have disappeared before August 15th. Such gradual melting will not create those spasmodic floods, common in the local streams below, that often do so much damage to irrigation work. The floods in all the large tributaries are held back by these causes, and, in the outcome, the

"June rise," about which so much has been said, will reach Cairo on an average one month later than usual. If the levees of the delta are of average strength, they will have taken care of the "spring rise" without any serious damage having been done. It will have passed down and out of the way of the "June rise," held back from causes before described, and the latter rise, from the same causes, will be of much less intensity than in former years, and will pass down to the Gulf and do but little damage.

The "June rise" is not caused entirely by melted snows, but is reinforced to some extent by local rains on the Lower Missouri and the Mississippi. The retarding of the melting snows of the "June rise" one month, by the means before described, will result in lessening its intensity to that amount, as the effects of the local rains will have passed down.

Reforesting the upper water-shed of the Missouri, as pointed out, is a work more expensive in the time required in its accomplishment than its cost in dollars and cents.

No plan has been suggested for the control of the floods of this mighty river which will not require years of time, patience, and labor. It is believed that the plans laid out by the Mississippi River Commission, as understood, are along conservative lines, as regards this great delta, that is, to hold the river to its present channel by the revetment of its concave or caving banks, by contracting its channel at bars by proper works, and, where necessary, by dredging these bars for navigation purposes. In addition to these works is considered (as the writer understands the views of the Commission) a substantially built system of levees on each bank. To carry out these plans will require much time and patience, with proportionately much labor and expense.

To make a permanent success of these works in the delta, it is considered essential that the high floods of the Ohio and Missouri water-sheds shall be kept apart. The only feasible means of accomplishing that desirable object is by impounding the flood-waters of the latter stream in the immense forest reservoirs at its head.

Reforesting the open areas among these forests, will be small in expense compared to the work in the delta. For a few years it will require the labor of as many men as can be worked to advantage for five months of each year, in resetting the pine saplings, and in making

the small ditches necessary for irrigating them. This will require about 1 000 men for a few years. After the shoots have taken root, fewer men, who should be expert at irrigation, will be required. Nature, with the warm rays of the sun, will do the rest. It is believed that within five years after this work is started its beneficial effects will be apparent.

To make a success of this reforesting will require the application of the laws of forestry suitable for each latitude of the North American Continent and to each species of pine found most suitable; and this must be combined with the experience of expert irrigators and much patience and care.

When it is considered that the successful accomplishment of this reforesting is one of the main factors, in conjunction with the contemplated works in the delta below, in adding to the resources of the Nation each year from $300 000 000 to $400 000 000, as estimated heretofore in a detailed report, it becomes a matter of national importance, and should be carried through regardless of whatever expense, within reasonable bounds, its accomplishment may require.

Closely allied to, and as one of the most important outgrowths of, forest preservation is the Reclamation Service, brought to the front by the broad-minded statesmanship of President Roosevelt and at once adopted by Congress. Its design is to reclaim, under a wisely guarded law, the arid regions of the mountain States on each slope of the Continental Divide, by expending the proceeds of the sales of the public lands in the construction of reservoirs and ditches leading therefrom for purposes of irrigation. One of the most direct benefits to the Nation from the construction of this reclamation system, when finished, will be the large annual saving in the production of the sugar beet, now so quickly and profitably changed into sugar, one of the necessaries of life. The belt of country lying between the 100th meridian and the Continental Divide and between New Mexico and the Canadian line, is eminently adapted to the production of the sugar beet. It is believed that the portion of this belt, under the influence of the Reclamation Service, on each slope of the Great Divide, when brought under proper cultivation, can produce enough sugar, in addition to what is now being produced, to take the place of the 2 500 000 tons now imported from foreign countries.

In other words, these millions of dollars now being spent on the

Reclamation Service will add each year to the resources of the Nation, when completed, at least $200 000 000, besides other benefits of great value which will naturally suggest themselves.

Bearing on the question of its influence in holding back the flood-waters of the streams making up the great "June rise," A. P. Davis, M. Am. Soc. C. E., Chief Engineer of the Reclamation Service, has kindly furnished the following table of the "Irrigating Projects" under construction in the Missouri River drainage basin:

Projects.	Acreage.	Discharge, in second-feet.	Storage, in acre-feet.
Huntley, Mont.	33 000	400	....
Sun River, Mont.	300 000	3 500	600 000
North Platte, Nebr.	300 000	3 500	1 000 000
Lower Yellowstone.	67 000	800	....
Milk River, Mont.	80 000	1 000	....
Williston, N. Dak.	40 000	500	....
Belle Fourche, S. Dak.	100 000	1 200	200 000
Shoshone, Wyo.	150 000	1 800	400 000
Totals.	1 070 000	12 700	2 200 000

The experience derived from irrigation on a small scale on the local streams of that water-shed is that the water which goes through the ditches and is turned out from them, is soaked up by the soil and does not return to the stream for a month or more. This happens in cases where the ditches are not more than ¼ mile from the stream. In the case of the above-named projects the main canals will be from 1 to 3 miles distant from the streams from which they flow. It can be estimated safely that the water passing out of the main canals of the above "projects" during May, June, and July, does not return to the streams from which they radiate for such a length of time that this discharge, together with the water impounded by the dams, can be considered as reducing the volume of the "June rise" to that extent. The volume of water thus held back amounts, by calculation, to about 4 500 000 acre-ft.; not a large volume, yet there are times when it might have a perceptible effect.

It must be borne in mind that a test has never yet been made as to the ability of the two earthen walls, comprising the levee system on each bank of the Mississippi between points opposite Cairo and to and

below New Orleans, to withstand the pressure that would be brought to bear, did the "spring rise" from the Ohio and the "June rise" from the Missouri combine near Cairo and attempt the passage to the Gulf. Judging the future by the past, the result, with the existing system of levees, would at least be problematical.

Neither the magnitude of the problem of the successful control of this river nor the benefits to be derived therefrom, are fully realized by the country. The history of engineering affords no precedent to serve as a guide in solving the problem. At flood tide its forces are irresistible when brought in direct antagonism to the puny work of Man. At times its actions appear almost as erratic as a child's. It has no respect for State's rights. In a few weeks time it will slice off a large area from one State and as quickly deposit it below on the opposite bank, to add to the territory of a neighboring State. It is only by humoring its eccentricities and guiding its inherent power, that Man can accomplish the solution of its control. This plan was successfully carried through at its mouth, when it was forced to dig out a channel 35 ft. deep (where a depth of 15 ft. had existed) across sand-bars formed by its own currents, by the construction of the jetties.

The cost of the work necessary for the control of the flood-waters will be great. The benefit to the United States arising therefrom by adding to its resources 4 000 000 bales of cotton and a proportionate amount of sugar each year makes it evident that its cost should be borne by the Nation and not by the comparatively few cotton and sugar planters of the delta, as has heretofore been done through State agencies. It should be evident that it is as much the province of the General Government to take care of the harbors of this vast inland sea and the channels leading to them as to take care of the harbors and channels of the seaboard and the Lake States.

As a means of keeping open this important channel of commerce, the building of levees, the revetment of the caving banks on their front, the contraction of its channel at low water in the interests of navigation, are the same means rendered necessary in the construction of the same works for the purpose of prevention of disastrous overflows. Any plan of river improvement for navigation without the aid of levees on the banks, would be a serious mistake and would result in failure. A bad crevasse in the levees would decrease the volume of water below, and counteract one of the governing ideas in the plans of

the Mississippi River Commission, viz., the contraction of the currents for scouring purposes.

The foregoing points out an additional reason why the cost of this great work should be borne by the Government. It carries with it the result that this entire system of work—the control of overflows by levees, and the works for the improvement of navigation—shall be under one central control. The two systems go hand-in-hand, and are interdependent.

To recapitulate: The solution of this great problem consists in keeping apart the "June rise" and the "spring rise," so that the latter floods will have passed before the advent of the former at the mouth of the Ohio.

As noted before, there is no relief to be expected from the Ohio. It is left, then, to the Missouri water-shed to furnish the means for the object required. The head-waters of this stream must be impounded in immense reservoirs, for such a length of time as will without doubt prevent the "June rise" from making its advent until the "spring rise" has passed.

Nature has constructed, as an object lesson, those immense storage reservoirs for protecting the winter's snows in the dense forests in and near the great Continental Divide. It directs us to reforest those large areas, which at one time were clothed with dense forests, but which have been denuded by fire and other unknown causes.

Nature teaches us that these open spaces, once clothed with forests, can be reclothed with similar forests if Man will use the means indicated by Science. Then will follow the slow melting of the conserved winter snows, thus keeping apart the "June rise" and the "spring rise," and, in conjunction with a substantial system of levees in the delta, this will afford the only solution for the permanent control of the periodical floods that sweep down through this great delta.

In the preparation of this paper, the writer is indebted to H. N. Pharr and W. I. Hardee, Members, Am. Soc. C. E., for information as to conditions in the Mississippi Delta subsequent to 1873, and to A. P. Davis, M. Am. Soc. C. E., of the Interior Department, and to Messrs. Kellogg, Bonsteel, Plummer, and Zoo, of the Agricultural Department, for valuable statistics.

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TRANSACTIONS

Paper No. 1102

ELECTRIC RAILWAYS IN THE OHIO VALLEY BETWEEN STEUBENVILLE, OHIO, AND VANPORT, PENNSYLVANIA.*

By George B. Francis, M. Am. Soc. C. E.

With Discussion by Messrs. F. Lavis, George B. Preston, J. Martin Schreiber, William J. Boucher, and George B. Francis.

There has been constructed during the past two years, along the northerly and westerly banks of the Ohio River, between Vanport, Pa., and Steubenville, Ohio, about 40 miles of double-track, standard-gauge, electric railroad, which affords this busy, thriving, industrial section a high-grade interurban road.

The lines form the connecting links uniting the East Liverpool, Wellsville, Steubenville, and Toronto systems, and, with their connections, at Vanport on the north and Steubenville on the south, make possible through travel by trolley between Pittsburg, Pa., and Wheeling, W. Va., except for a short piece of track, now under construction, in the vicinity of Sewickley.

The configuration of the country through which the road has been built is such as practically to preclude the location and construction of a future competing line.

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* Presented at the meeting of January 6th, 1909.

The railways described have been built by three constituent companies:

The Steubenville and East Liverpool Railway and Light Company.—This company owns and operates the properties of the Steubenville Traction and Light Company and the Toronto Electric Light and Power Company. The former of these acquired properties owned and operated the street railway system in Steubenville, and a single-track line to Toronto on the north, a distance of about 10 miles. The new company has reconstructed this single-track road, improving its alignment and grade at various places by locating the tracks on a private right of way; has added a new second track between Steubenville and Toronto, and has built a new double-track road between Toronto and Wellsville, a distance of 7.86 miles. The company also furnishes light and power for commercial purposes in Steubenville and Toronto. At Steubenville, the southerly terminus of the system, connection is made with the Wheeling Electric Railway, which reaches Wheeling, W. Va., a distance of about 21 miles, also with the Steubenville and Brilliant Line, on the west side of the river.

The East Liverpool Traction and Light Company.—This company serves the East Liverpool district, which extends eastward as far as the State line between Ohio and Pennsylvania, and southward to the southerly limits of Wellsville, and includes the street railway system in East Liverpool, a branch line across the river to Chester, W. Va., and a 3-mile spur track to the company's coal mine up Island Run. It also furnishes light and power for commercial purposes in East Liverpool, Chester, and Wellsville. The initial move in improving and increasing the transportation facilities demanded by this territory was made by this company. The road was originally a single-track line, lying wholly in the highways and streets, and had several dangerous steam railroad crossings at grade. In the reconstruction of that portion of the system which forms a part of the through main line, provision was made for a double track, which has been laid partly in new location, and as far as practicable on private right of way, improving the grade and alignment, and eliminating the grade crossings.

The major portion of this property was reconstructed during 1905 and 1906, thus preceding the construction of the stretches of road southward to Toronto and Steubenville, and northward from East Liverpool to Vanport, which has resulted in the continuous line of about 40½ miles of double-track road described herein.

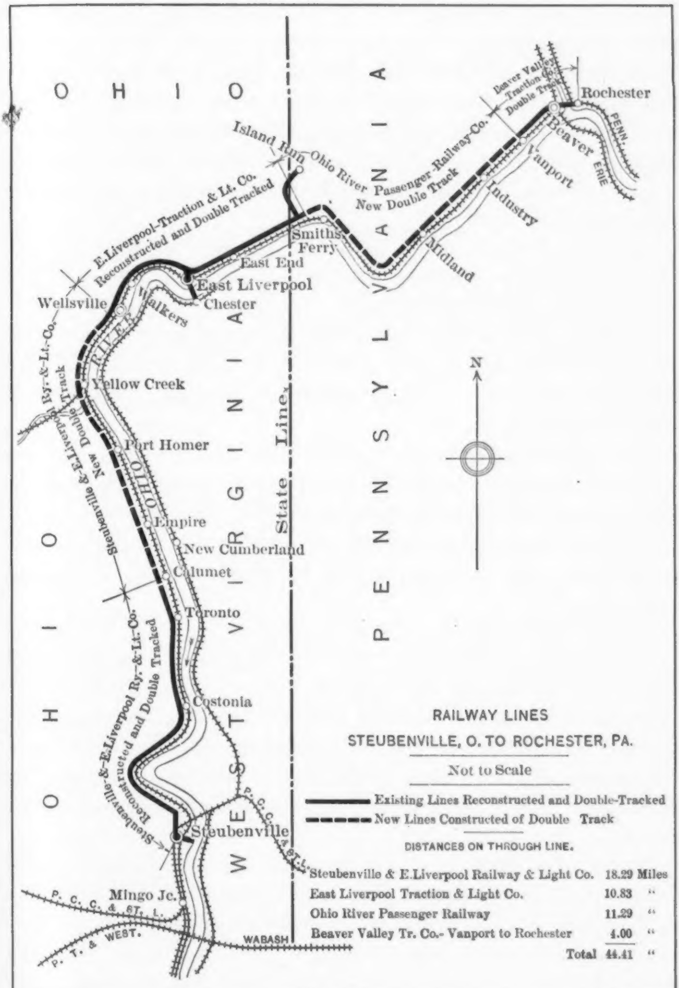

OHIO
OHIO
WEST VIRGINIA
PENNSYLVANIA
State Line
N
Island Run
Ohio River Passenger Railway Co.
New Double Track
E. Liverpool Traction & Lt. Co.
Reconstructed and Double Tracked
Beaver Valley Traction Co. Double Track
Steubenville & E. Liverpool Ry. & Lt. Co.
New Double Track
Steubenville & E. Liverpool Ry. & Lt. Co.
Reconstructed and Double Tracked
Rochester
Beaver
Vanport
Industry
Midland
PENN.
ERIE
Smiths Ferry
East End
East Liverpool
Chester
Wellsville
Walkers
RIVER
Yellow Creek
Port Homer
OHIO
Empire
New Cumberland
Calumet
Toronto
Costonia
Steubenville
Mingo Jc.
P. C. C. & ST. L.
P. T. & WEST.
WABASH
RAILWAY LINES
STEUBENVILLE, O. TO ROCHESTER, PA.
Not to Scale
Existing Lines Reconstructed and Double-Tracked
New Lines Constructed of Double Track
DISTANCES ON THROUGH LINE.
Steubenville & E. Liverpool Railway & Light Co. 18.29 Miles
East Liverpool Traction & Light Co. 10.83 "
Ohio River Passenger Railway 11.29 "
Beaver Valley Tr. Co.- Vanport to Rochester 4.00 "
Total 44.41 "

FIG. 1.

The Ohio River Passenger Railway Company.—This company has built the new double-track road between East Liverpool (where connection is made with the lines of the East Liverpool Traction and Light Company) and Vanport, a distance of 11.29 miles. At the latter place connection is made with the lines of the Beaver Valley Traction Company, with which company a traffic agreement has been made for operating cars into Rochester, Pa. Arrangements are being perfected with the Pittsburg and Lake Erie Railroad for a connection at Beaver and for the sale of tickets for through travel to any point on either system.

A tri-party agreement has been executed with the foregoing constituent companies permitting the joint use of the main-line trackage between Steubenville and Rochester, a distance of 44½ miles.

Location.—The portion of the Ohio Valley traversed by these connecting lines is narrow, with high and, in some places, steep and precipitous hillsides and bluffs alternately advancing and receding from the shore of the river.

The location along the bank of the river on a low grade line was practically pre-empted by the Cleveland and Pittsburg Division of the Pennsylvania Lines, and it required considerable engineering study and skill to secure private right of way, and locate a line which would not only give easy curves and grades, but avoid expensive trestles and viaducts.

Railway Construction.—The permanent way consists of a graded roadbed, located partly in the streets and highways, but for the greater portion of the distance on a private right of way, laid with 85-lb. rails, and ballasted in a first-class manner. The alignment and grades are such as are compatible with high-grade urban and interurban roads. All streams and waterways are crossed by substantial bridges and culverts, the former being of steel and the latter of concrete or stone masonry.

Track.—The track is of standard-gauge construction, and in the main is of 85-lb. T-rail, Am. Soc. C. E. section, laid in 60-ft. lengths on 6 by 8-in. by 8-ft. hardwood ties, mostly oak, spaced 24 in. from center to center, and ballasted with gravel or broken stone. The rails are bonded for the return circuit with single 000 protected bonds, except in East Liverpool, where double bonds of the same type are used on each joint. Cross-bonds connect the rails at intervals of 1 000 ft.

PROFILE OF GRADE LINE BETWEEN VANPORT, PA. AND STEUBENVILLE, O.

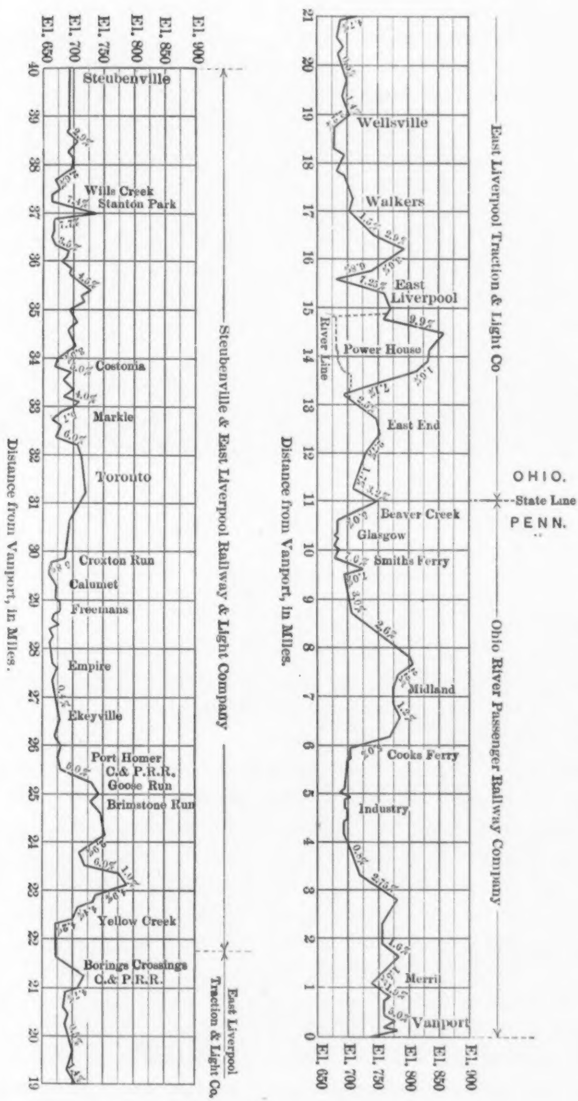


Fig. 2.

The tracks are generally 9 ft. 8½ in. from center to center; but where bridges or trestles are crossed, provision is made for spacing the tracks 10 or 12 ft. from center to center. Spring frogs of standard pattern are used in all cases outside of city streets. Fig. 3 shows the standard adopted for roadway construction, outside of towns and cities, both in cuts and fills.

Grading.—Outside of cities and towns the grading was practically side-hill work, necessitating some heavy cuts and fills. In many instances the railroad right of way parallels the highway which was originally cut in the sides of these precipitous hills. This roadway was widened, providing room for both the highway and the double-track railway line.

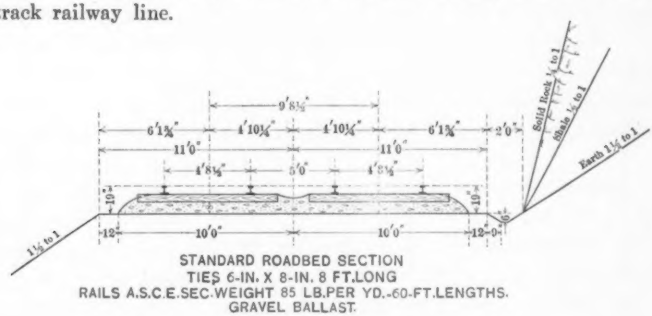


Fig. 3.

The work required the excavation of about 750 000 cu. yd. of material of all classes, a considerable percentage being rock, mostly shale and sandstone.

Beginning at Seventh Street, in Steubenville, the principal work may be described, in sequence toward Vanport, as follows: Grading and paving in highway for track and street widening for 2.76 miles; thence grading and culverts for double track on private right of way across the King and other property for 2.23 miles, this embracing considerable earth and rockwork; thence track grading and highway paving and widening for 1.42 miles; thence earthwork grading and bridging on private right of way through the Toronto Realty Company's property for 0.30 mile to the streets of Toronto; thence grading for track in the streets of that town for 2.2 miles; thence on private right of way and over a double-track, timber trestle at Croxton's Run, 360 ft. in length, for 0.40 mile; thence grading for track, partly in

FIG. 1.—RETAINING WALL, COOKS FERRY, OHIO.

FIG. 2.—YELLOW CREEK BRIDGE.

streets, but mostly on private right of way through Calumet, Empire, and Ekeyville for 4.07 miles to the north end of the steel viaduct, at Port Homer, over the Cleveland and Pittsburg Railroad, 450 ft. long; thence side-hill grading on private right of way, including relocation of highways along what is known as The Narrows, to Yellow Creek, for 3.01 miles, this stretch including steel viaducts at Goose Run, 335 ft. long, Brimstone Run, 175 ft. long, various cattle passes, heavy retaining walls, and small bridges; thence by a single-span steel bridge, 174 ft. long, over Yellow Creek; thence under the Cleveland and Pittsburg Railroad through existing bridge openings, and side-hill grading, including the straightening and paving of the highway for 0.78 mile to Borings Crossing, a new bridge being there constructed for the Cleveland and Pittsburg Railroad over the new road and highway, thus removing an existing grade crossing; thence track grading and street paving in Wellsville, for 3.69 miles to the city line of East Liverpool, this stretch including the Wellsville steel viaduct, about 300 ft. long; thence track grading and street paving in East Liverpool for 7.14 miles to the easterly city line, this stretch including the Jethro Run steel viaduct, about 475 ft. long, the Sixth Street steel viaduct, 664 ft. long, Bradys Run culvert, Dry Run bridge, and other structures. For a considerable portion of the latter distance the highways were widened, and some of the new railroad was placed on a private right of way; thence crossing Beaver Creek by a two-span double-track steel bridge, 306 ft. long, and crossing under the Cleveland and Pittsburg Railroad through an existing opening, and in streets for 1.47 miles to a bridge constructed over the Cleveland and Pittsburg Railroad tracks, for both the new railroad and the highway, thus removing a grade crossing; thence track grading on a private right of way, and straightening, widening, and paving the highway for 2.20 miles to Midland; thence track grading and paving for 1.51 miles through this town; thence grading on a private right of way for 1.17 miles to Industry; thence track grading and street paving through this village for 0.37 mile; thence grading on private right of way (mostly side-hill work) for 4.55 miles to the end of the new double track at Vanport. This last stretch embraces the following steel viaducts and other important structures: Four Mile Run viaduct, about 323 ft. long; Barclay Run viaduct, about 275 ft. long, and Vanport trestle, about 700 ft. long.

The side-hill grading was quite difficult, as a great number of slides came down the hillsides, in some instances from considerable distances, causing large expense for their removal. The clayey nature of the material, combined with water from the hills, was the cause of many of these slides. The excavated material consisted of earth, a mixture of earth and rock fragments of varying sizes, and solid rock.

The side-hill work necessitated the construction of a number of retaining walls at different points, the principal ones being at The Narrows, just south of Yellow Creek, and one near Cooks Ferry. The former is about 350 ft. long and has a maximum height of 30 ft.; each contains approximately 1 600 cu. yd. of concrete. The wall at Cooks Ferry is of similar dimensions.

Paving.—The line passes through fifteen communities, and in nearly every instance where a franchise was given for the use of streets or highways unusual requirements for paving were exacted. In some instances the entire width of the street was paved; in others, where the street grade was changed, the embankment slopes in front of residences were paved, as well as the street surface.

As this is a great brick manufacturing center, all paving consisted of vitrified brick on a sand cushion, 11 miles of track being thus paved, the cost averaging approximately $0.90 per sq. yd.

Bridges.—All important bridges, with the exception of those over Beaver and Yellow Creeks, are deck structures of the viaduct type, weighing about 700 lb. per lin. ft. of double track, and were calculated for a concentrated load of 24 tons on two axles at 10-ft. centers, or a uniformly distributed load on trusses of 1 800 lb. per lin. ft. of each track.

The standard of construction calls for 35-ft. braced towers with battered posts, and 60-ft. spans between the towers, with provision for expansion at each tower. All columns and towers are fixed to the foundations with long anchor-bolts set in the piers, and at the abutments the structure is held in place by short bolts set after the erection of the steel, thus allowing for creeping during erection. The structures, though light, were built for interurban electric passenger service only, and are thoroughly braced and very rigid. The floor systems are stiffened further by carrying the ties clear across the structure. All foundations and abutments are of concrete. Fig. 4 is a typical design for these viaducts.

FIG. 1.—TRACK VIEW AT INDUSTRY, PA.

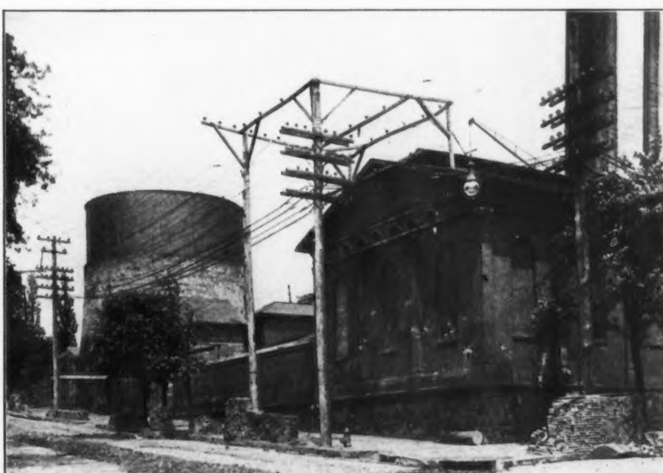


FIG. 2.—WATER-COOLING TOWER, STEUBENVILLE POWER PLANT.

The bridges over Beaver and Yellow Creeks are double-track, through, two-truss structures, having rigid members with riveted connections. The load is distributed evenly over the bearings by using pin-connected shoes. At the expansion end the shoe rests on a nest of turned rollers which are free to move in a planed roller box bolted to the masonry. The structures are secured firmly to the concrete abutments and piers by long anchor-bolts set in the masonry.

The Beaver Creek bridge consists of two double-track, through spans, each 153 ft. long; the Yellow Creek bridge consists of one double-track, through span, 174 ft. long. The steel for these bridges was fabricated and erected by the Fort Pitt Bridge Works.

At Sixth Street, East Liverpool, a steel viaduct was built over what is locally known as Horn Switch. It is 664 ft. 4 in. over all, is 40 ft. above the foundation at its highest point, and weighs about 233 tons. The steel for this bridge was fabricated by the American Bridge Company, and erected by the Cleveland Engineering Company.

The aggregate weight of all the new truss bridges and viaducts approximates 1 100 tons.

A number of small I-beam bridges were built, and three of the existing highway structures were strengthened to take care of the loading imposed by the addition of a second track.

Power Requirements.—The power requirements of the original companies forming a part of this property were provided for in four generating stations, located at East Liverpool, Wellsville, Toronto, and Steubenville. These stations supplied current, not only for the street railway operations, but also for municipal lighting and power. To this load factor were added the requirements for the operation of about 60 miles of new track and an increase of 50% in the lighting loads at East Liverpool, Steubenville, and elsewhere.

In the changes, alterations, and additions to the generating, distributing, and transmitting system, the existing houses and equipment were utilized as far as possible.

After a careful study of the conditions, it was decided to consolidate all the generating equipment in the East Liverpool and Steubenville power-houses. The loads formerly carried by the Toronto and Wellsville stations were transferred to these main stations, and the buildings were remodeled and equipped as sub-stations, one new substation being built at Industry.

The difficulties of this work were increased by the necessity of keeping the generating stations in continuous operation while the changes and alterations were being effected. The work was accomplished successfully without the interruption of any service that was being rendered.

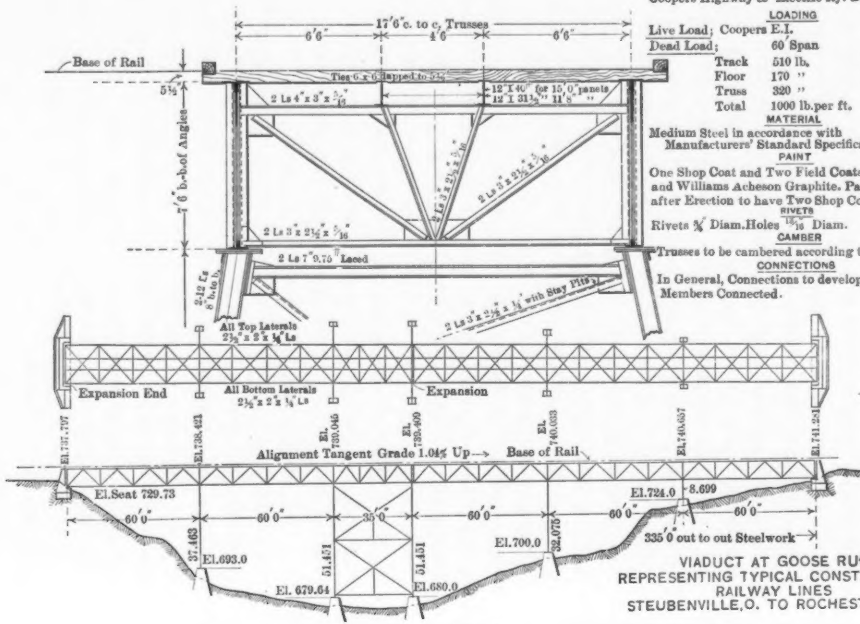
East Liverpool Power-House.—The original building was a brick structure with a wooden floor and a tar and gravel roof on timber supports. The old wooden floor was taken out and replaced with reinforced concrete, giving increased cleanliness and greatly reduced fire risk.

The load on this station was increased by the addition of that formerly carried by the Wellsville station and the new Industry substation, as well as by the increased requirements of the railroad, to provide for which the alternating-current generating equipment was increased by the addition of one 1 000-kw. turbo-generator and one 500-kw. turbo-generator, and the direct-current equipment by one new 300-kw. rotary converter and the transfer of a 300-kw. engine-type generator from the Steubenville power-house. A new 7-panel switch-board was installed to take care of this additional electrical apparatus.

The turbines as well as the alternating-current, belt-driven generators at this plant produce 60-cycle, 2-phase current at 2 200 volts. To reduce this voltage for the operation of the rotary converter, two 175-kw. step-down transformers were installed, and to raise it for transmission to Wellsville and Industry, six 175-kw. step-up transformers were provided. Excitation is taken care of by one steam- and one motor-driven exciter, either of which is of sufficient capacity to furnish excitation for all the alternating-current generators. A Tirrill regulator was installed for voltage regulation.

The boiler plant was enlarged by the addition of a wooden frame structure, sheathed with corrugated iron, in which were installed four 500-h.p. Stirling water-tube boilers. A 2 000-h.p., open feed-water heater, and two 12 by 7 by 12-in. boiler feed pumps were also provided.

In order to operate the turbines, a barometric condenser with dry vacuum pump and two motor-driven circulating pumps were installed. Water for condensing purposes is taken from the Ohio River, and, as there is about 40 ft. difference between the high and low stages of the



SPECIFICATIONS

Coopers Highway & Electric Ry. Bridges (1901)

LOADING

Live Load; Coopers E.I.

Dead Load;	60' Span	35' Span
Track	510 lb.	510 lb.
Floor	170 "	160 "
Truss	320 "	230 "
Total	1000 lb. per ft.	900 lb. per ft.

MATERIAL

Medium Steel in accordance with Manufacturers' Standard Specifications.

PAINT

One Shop Coat and Two Field Coats of Sherwin and Williams Acheson Graphite. Parts inaccessible after Erection to have Two Shop Coats.

RIVETS

Rivets ⅞" Diam. Holes 15/16" Diam.

CAMBER

Trusses to be cambered according to Specifications.

CONNECTIONS

In General, Connections to develop the Members Connected.

VIADUCT AT GOOSE RUN
REPRESENTING TYPICAL CONSTRUCTION
RAILWAY LINES
STEUBENVILLE, O. TO ROCHESTER, PA.

Fig. 4.

river, unusual means had to be provided for elevating the water to the condensers. To this end a vertical intake or well of reinforced concrete was built on the bank of the river, extending from below the low-water to above the high-water level. Vertical centrifugal pumps were then installed at the bottom of this well and connected by vertical shafts to two 50-h.p. 2 200-volt induction motors placed above the high-water mark. From the bottom of the head-house a 24-in. intake pipe was extended to the center of the channel of the river, terminating in a timber crib.

The coal supply for this plant is obtained from the company's mine, from which it is carried over a 3-mile branch line and delivered in dump cars at the fire doors of the boilers. The generating capacity of the station has been increased from 1 200 to 3 000 kw.

Steubenville Power-House.—This is a brick structure with a brick floor and a slate roof, supported by steel trusses. A separate building, of similar construction, was enlarged to provide for the installation of two 500-h.p. boilers.

The station load was augmented considerably by the transfer of the lighting load formerly carried at the Toronto station, the increased lighting load for Steubenville, and the additional power required for the operation of the 25 miles of newly constructed railway lines which it serves. To take care of these requirements, the following additional equipment was installed: Two 500-kw. turbo-generators, producing 60-cycle, 2-phase current at 2 200 volts; two 175-kw. and two 100-kw. step-up transformers for raising the voltage to 13 200 volts, 3-phase, for transmission to the Toronto sub-station. The switch-board was enlarged by the addition of four new panels to handle this new equipment. Excitation is taken care of by one steam-driven and one motor-driven exciter, either of which is of sufficient capacity to furnish excitation for all the alternating-current generators. A Tirrill regulator was also installed.

The plant was enlarged further by the addition of two new 500-h.p. Stirling boilers, and the necessary condensing apparatus for the turbines, consisting of a barometric condenser, dry vacuum pump, and engine-driven centrifugal and circulating pumps.

As the only source of water supply for condensing purposes was the city water system, it was necessary to construct a cooling tower and an additional engine-driven centrifugal pump to deliver water

thereto. In erecting this tower, use was made of an old gas tank and well on the property.

The capacity of this station has been increased from 1 800 to 2 500 kw.

Toronto Sub-Station.—The Toronto sub-station building formerly contained the generating apparatus supplying current for the arc- and incandescent-lighting systems of Toronto, and was built of tile blocks, with a brick floor and a slate roof. As now equipped, it contains two 300-kw. rotary converters furnishing direct railway current at 585 volts, and operated from the transmission line through six 110-kw. step-down transformers, and two 110-kw. transformers which deliver 2-phase current at 2 200 volts for the incandescent-lighting circuits, and for the arc-lighting circuits by constant-current transformers and mercury arc rectifiers. The station has a 10-panel switch-board.

Current is received from the Steubenville power-station at 13 200 volts, over two 3-phase transmission lines 9¼ miles long.

Wellsville Sub-Station.—The apparatus for this sub-station is installed in a building which was formerly the Wellsville power-station, supplying Wellsville with arc lights for street lighting and incandescent lights for commercial purposes, as well as direct current for street railway operation.

The building is of brick, with a solid concrete floor and a tin roof. Current is received at 6 600 volts from the East Liverpool power-house, over two 3-phase transmission lines each 4¾ miles long.

The present sub-station apparatus is designed to take care of the commercial and railway loads, and consists of two 300-kw. rotary converters furnishing current at 585 volts for railway purposes, the current being delivered to the rotaries from the transmission line through six 110-kw. step-down transformers; and four 75-kw. transformers which deliver 2-phase current at 2 200 volts for the incandescent-lighting circuit. The arc lights are taken care of by two constant-current transformers supplied by the 2 200-volt bus. The station also contains a 200-kw. "balancer set," furnishing direct current at 500 volts for a special commercial circuit, supplying shop motors which could not be operated at the railway voltage. The station has a 9-panel switch-board.

Industry Sub-Station.—This sub-station is a new brick structure, with a concrete floor, and a reinforced concrete roof supported by

I-beams. It contains two 300-kw. rotary converters which receive current from the transmission line through six 110-kw. step-down transformers and deliver direct current at 585 volts for railway service only. Current is transmitted to this station at 13 200 volts from the East Liverpool station over a single 3-phase transmission line about 10 miles long. A 5-panel switch-board is installed.

Overhead Construction.—The usual type of span-wire construction used by direct-current interurban roads was adopted. The poles are of chestnut, from 30 to 35 ft. long, with 7-in. tops. They are set 6 ft. in the ground, with head and breast boards, and are given a deflection away from the tracks of 18 in. in 24 ft. The poles are approximately 100 ft. apart, with a minimum clearance of 24 ft. from center to center of poles, measured at the track level. The poles on one side of the road are 5 ft. higher than on the other, and carry, in addition to the trolley, the feeder and transmission lines. The butts of all poles carrying high-tension wires are charred for a distance of about 18 in. above and below the ground line. The tops and gains of all poles were painted with two coats of heavy black asphaltum before erection and before the attachment of the cross-arms. The cross-arms are painted yellow pine, fitted with locust pins, secured to the poles by through galvanized-iron bolts, and braced with galvanized-steel braces.

The trolley lines are mainly of 00 hard-drawn copper wire supported by $\frac{3}{8}$-in. galvanized stranded span wire to which they are attached by round-top hanger suspensions with 15-in. bronze soldered ears, except on curves, where the ears are carried by gooseneck suspensions with wood strain insulators. They have a clear height above the track of 19 ft. 6 in. outside of city limits and on private right of way, while in the cities this clearance is increased to 21 ft. 6 in. Protection from lightning is furnished by arresters placed on the poles at intervals of about 1 000 ft.

Transmission and Feeder Lines.—The six wires forming the double 3-phase transmission lines between East Liverpool and Wellsville are supported on one 8-ft. cross-arm, the wires of each circuit being 16 in. apart. The double transmission lines between Steubenville and Toronto are carried in delta on two cross-arms, with the apex of the triangle at the bottom, the cross-arms being spaced so that the wires form a 24-in. equilateral triangle. Between East Liverpool and In-

dustry, the single 3-phase transmission line is carried on a 7-ft. cross-arm, the wires being spaced 24 in. apart. This construction makes possible, with the least alteration, future provision for a double 3-phase line, similar to the Steubenville and Toronto lines, by simply adding another cross-arm with two insulators.

The wires are carried on brown porcelain, high-tension insulators, 5⅝ in. in diameter and 4¼ in. in height, and are protected from lightning by suitable lightning arresters and choke-coils in each of the power-houses and sub-stations. The lines consist of No. 2 B. & S. hard-drawn copper wire, except between East Liverpool and Wellsville, where No. 4 wire is used. This wire is bare, except through built-up districts, where it is protected by water-proof insulation. At two points on the route the transmission lines are carried on independent pole lines which have been erected for short distances on a private right of way.

In addition to the direct-current railway feeders required for local service, the main line of the road is taken care of by 795 000-cir. mil. aluminum feeders carried on a 2-pin cross-arm below the transmission line, taps to the trolley being provided at intervals of 1 000 ft.

Cars.—New rolling stock has been purchased for the through service between Steubenville and Rochester, and consists of 18 double-truck Pullmans, 44 ft. long, finished in mahogany, with high-backed, red plush seats, having a seating capacity of forty-four. Cars of this type, and for service of this character, are usually fitted for greater carrying capacity, but in this equipment four seats, two on either side, were omitted, the seats being respaced to give more liberal distance between them, following more nearly, in this particular, steam railroad practice.

Each car is equipped with four 60-h.p. motors, mounted on Brill trucks with steel wheels. A smoking compartment, with a seating capacity of sixteen, is provided at one end; all cars are heated and lighted by electricity, and fitted with electric arc headlights, Lecturn signal lights, electric bells, parcel racks, and other conveniences.

The exterior finish is a rich yellow, lettered in aluminum outlined in black. On the letter-board over the windows, the cars bear the words "Ohio Valley Scenic Route."

Organization in the Field for Roadway Construction.—The engineering of those portions of the roadway of the East Liverpool Traction and Light Company's property constructed and reconstructed during 1905 and 1906 was looked after by Westinghouse, Church. Kerr and

Company. The work was executed under various contracts at unit prices by others and by the company's own forces.

The engineering of the subsequent roadway extensions of this company's property, the construction and reconstruction on the Steubenville and East Liverpool Railway and Lighting Company's property, and the work on the Ohio River Passenger Railway Company's property constructed during 1907 and 1908 was looked after and constructed directly by the forces of Westinghouse, Church, Kerr and Company, as was also the power work during 1907 and 1908.

The roadway organization in the field consisted of the engineer in charge, assisted by field parties and superintendents on the different main divisions, with sub-divisions under general foremen, walking bosses, foremen of gangs, etc., these being revised from time to time as the work demanded in its several parts, such as grading, masonry, track-laying, paving, and overhead work. Some unavoidable difficulties in obtaining right of way delayed the work so that the time covered was more than would have been necessary otherwise. The maximum number of men employed at one time was about 1 000. No steam shovels were used or required. Four locomotives, with about 30 dump cars, were utilized more or less continuously in the grading and in the distribution of track material and ballast. There were various concrete mixers, derricks, and pile drivers. The total cost of the roadway work for 1907 and 1908 was about $1 600 000. The cost of the power work done during 1907 and 1908 aggregated about $450 000.

Government Work.—Extensive river improvements are being made by the United States Government in the construction of a deep-water channel between Pittsburg and Cairo and the building of dams and locks which will make navigation possible on the Ohio throughout the year. It is estimated that at the present time, with these improvements still far from completion, more than 10 000 000 tons of freight per annum are moved in boats and barges during the high-water stages, over the 40 miles of river between Beaver and Steubenville.

Population.—Table 1 gives the approximate population of the various cities, towns, and districts through which the road passes, or which, by reason of connections made, may fairly be considered as contributory; also the approximate mileage.

Industries.—The section of the Ohio River between Beaver and Steubenville is rich in resources—coal, natural gas, oil, clay, quarries,

etc.—which make it a natural industrial center. Here are situated some of the large iron, steel, and tin-plate works of the United States Steel Corporation and a number of independent companies; extensive plants for the manufacture of vitrified sewer pipe, fire brick, paving brick, tile and other fire-proofing materials, as well as glass-works and paper mills.

TABLE 1.

District.	Mileage.	Population.
Pittsburg	8	700 000
Intermediate	17	30 000
Beaver	4	46 000
Intermediate	11	8 200
East Liverpool	11	44 500
Intermediate	10	18 900
Steubenville	8	41 500
Intermediate	19	11 500
Wheeling	14	100 000
	102	1 000 600

East Liverpool is the principal seat of the pottery industry in the United States. Three of the twenty-seven potteries located here are said to be the largest in the country; and this industry, which alone employs 12 000 hands, with a weekly payroll of more than \$100 000, produces more semi-porcelain tableware, door knobs, and porcelain electric fixtures, than any other city in the world.

Other important manufacturing interests, related or semi-related to the foregoing, make this 40 miles of river valley "a veritable hive of industry."

Amusement Park.—One of the most beautiful and popular amusement resorts in this section of the country is Rock Spring Park, at Chester, W. Va., just across the river from East Liverpool, and reached by one of the company's branch lines. It constitutes the main pleasure resort in this part of the valley, and no expense has been spared to maintain the high standard of its attractions. It is owned by the East Liverpool Traction and Light Company, and is operated by an amusement company to which it is leased.

Scenic Attraction.—The road passes through an exceptionally picturesque country, and gives promise of excellent possibilities for the development of a large excursion business; it will undoubtedly prove an attraction to pleasure seekers.

For its entire length, the route follows the river, and is located generally on the table lands affording many fine views of the O-hi-yu

or "Beautiful River," which fully justifies the name given to it by the Indian tribes formerly inhabiting its banks.

In the vicinity of Toronto, great apple orchards cover the hills for almost 10 miles, and in the spring the great mass of white and pink bloom against the fresh green of the hillside presents a strikingly beautiful picture. The beginning of the apple industry is traced to the eccentric woodsman, John Chapman, familiarly known as "Johnny Appleseed," who, during the latter part of the eighteenth century, devoted his life to planting apple trees in this vicinity.

Operation.—The various sections of the road were placed in operation as fast as completed, as follows:

Double track between Wellsville and East Liverpool, Sept. 15th, 1906.
Between Calumet and Port Homer, Dec. 14th, 1907.
*Between Steubenville and East Liverpool, Feb. 11th, 1908.
Between East Liverpool and Smith's Ferry, Feb. 29th, 1908.
Between Smith's Ferry and Midland, March 21st, 1908.
Between Steubenville and Calumet, June 30th, 1908.
The entire road between Steubenville and Vanport, July 1st, 1908.

Cars are run on 30-min. schedule on the main through line, with extras and specials as required.

Financiers, Engineers, and Constructors.—The construction of these railway lines has been financed by the Ohio Valley Finance Company, the Hon. W. Caryl Ely, of Buffalo, N. Y., President. The general supervision of the whole project, the securing of right of way, and the purchase of rolling stock was looked after by Mr. Edward McDonnell, of East Liverpool, Ohio, Assistant Treasurer, and Mr. Van Horn Ely, of Steubenville, Ohio, Vice-President of that company.

Westinghouse, Church, Kerr and Company, of New York City, were the engineers of construction and reconstruction of the old properties, and the engineers and constructors of the new properties, the roadway construction being under the supervision and direction of the writer assisted by Edwin J. Beugler, M. Am. Soc. C. E. The execution of the work in the field was under the direction of William V. Polleys, M. Am. Soc. C. E., Resident Engineer. Mr. George B. Preston was the engineer in charge of the mechanical and electrical work in the field and office.

*A part of the distance between Steubenville and Calumet was operated as a single-track road until June 30th, 1908.

DISCUSSION.

F. Lavis, M. Am. Soc. C. E.—The valley of the Ohio River and the territory tributary to it have witnessed a greater development of long-distance interurban electric railroads than any other part of the United States. Many of these lines run through sleeping, dining and drawing-room cars, built by the Pullman Company, of only slightly lighter construction than the standard equipment of the largest trunk lines of steam railroads. A description, therefore, of one of these lines, completed only about six months ago by one of the most prominent engineering firms identified with work of this kind, is presumably a description of the very latest and best practice in the construction of railroads of this type at the present time. Mr. Lavis.

The general characteristics of these interurban lines are, that they pass through the various cities and towns along the route on the surface of the streets, and as near the business centers as possible, while in the open country between, they are located, for the greater part of the distance, on private right of way, in many instances having numerous highway crossings at grade. This latter feature is not mentioned by the author, and may have been avoided on this road, but the number of these crossings, which are only slightly, if at all, less dangerous than those of steam railroads, is becoming increasingly large, even in States where a great deal of public money has been, and is being, expended in eliminating such features on steam railroad lines.

This road, between Rochester and Steubenville, passes through a territory described by the author as a very "hive of industry," and the population tributary to it, amounting to more than a million people, or practically 10 000 per mile on the whole length between Pittsburg and Wheeling, is composed largely of the class of prosperous mechanics and laborers which furnishes a larger patronage, in proportion to the total population, to a road of this kind than does any other. The most substantial construction, therefore, is apparently warranted.

In order to get some idea of the relative importance of the construction on such a line, it seems desirable to have some basis of comparison with other standard railroad practice. The bridges on this line are designed for a concentrated load of 24 tons on two axles at 10-ft. centers, or 1 800 lb. per lin. ft. of single track. The heaviest equipment of the New York Subways and Elevated Lines has a load of 30 tons on the two axles with the motors, and 11 tons on the other two, or an average of 2 080 lb. per lin. ft. of track, which is only slightly greater than that used on this line. The largest Pullman cars in use on steam railroads are about 80 ft. long, and weigh about 60 tons, this load being carried on two six-wheeled trucks, giving about 10 tons per axle and about 1 500 lb. per lin. ft. of track. A passenger locomotive of the Atlantic type, weighing about 75 tons, which is a fairly

Mr. Lavis. large engine for express passenger service on anything but mountain grades, has a load of about 20 tons per axle on the driving wheels, and the load, for engine and tender loaded, will be about 4 000 lb. per lin. ft. of track. A consolidation freight engine, weighing about 100 tons, has a load of about 22 tons on the driving axles, and will average 5 000 lb. per lin. ft. of track. The bridges on this road, therefore, will carry almost any standard railroad equipment except locomotives, the weights of the heavier types of which in ordinary use would be about double that for which these bridges were designed.

Having in view, therefore, the fact that the requirements of roads of this class are not, in many respects, very far removed from standard steam railroad practice on first-class lines, and are in some respects greater than the requirements on many inferior lines, it has seemed to the speaker that it would greatly enhance the value of the paper if the author would give some of the reasons for the very radical departures from steam railroad practice as regards the location of this line.

The author states that "It required considerable engineering study and skill * * * to locate a line * * * which would give easy curves and grades," considering the natural difficulties of topography, and the possibility of securing private right of way. Nothing is stated in the paper in regard to what the author considers to be easy curves for a road of this type, although, as a matter of fact, outside of the cities, curves of from 200 to 300 ft. radius, provided there were not too many of them, and the amount of central angle covered was necessary, would probably have little effect either on cost of operation or speed. In passing through cities, curves of as small a radius as 50 ft. are not infrequent. This difference between roads of this type and trunk-line railroads where a curve of 1 000 ft. radius is considered quite sharp and necessitates slowing down to from 20 to 25 miles an hour, is quite marked, and is due of course to the absence on the electric road of the long rigid wheel base of the steam locomotive.

The question of grades on these electric roads, however, is one about which far less is known, at any rate publicly. Railroads building new lines to be operated by steam, even in a great deal of the, as yet, undeveloped parts of the country, are very reluctant to adopt higher rates than 0.6% unless the topography of the country absolutely demands it, whereas on the profile of the road under consideration, as shown by Fig. 2, one grade of 9.9% is shown for a fairly long stretch, and there are several stretches of 6% to 7%, although most of them seem to be fairly short.

The reason, of course, for the badly broken grade line is the fact that the river bank is occupied by the steam railroad, thus forcing the electric road in many cases high up on the side-hills where these

come close to the river and necessitating its descent nearly to the river level where the hills lie farther back, in order to avoid a long detour to keep up. This characteristic of the country is shown very clearly in the view of the Yellow Creek Bridge. Mr. Lavis.

The ability to ignore, to a very large extent, considerations which affect the determination of the rates of grade, which govern steam railroad practice, is the chief characteristic which differentiates the location of interurban electric railroads, or any electrically operated railroad where single cars or multiple-unit trains are to be used exclusively, from that of those operated by steam or electric locomotives at the head of the train, although few data have as yet been made public as to the actual conditions which govern the determination of the most economical grades to be used on these electric roads.

On a steam railroad, in conjunction with the necessity of low grades, it is also almost as important to arrange the grade line so that the demand on the locomotive will be as nearly even as possible throughout comparatively long stretches. No considerations of this kind apparently influenced the fixing of the grades on this railroad, and here again it would undoubtedly be of considerable interest to know just what considerations governed the judgment of the engineers in adopting the grades they did on this particular road.

As the steam locomotive is most economically operated when the demand on the machine is fairly consistent, so also is this the case with almost any machine or power plant, including, and perhaps especially so, electrical power plants.

In the case of the latter, when power is being manufactured for the operation of a railroad, it may be considered that, where a fairly large number of cars are being operated, the demand will be equalized by the fact that some cars will be going down grade while others are going up, some stopped while others are going at full speed, etc.; but, on a road such as the author describes, where the service is infrequent and the number of cars in use at a time small, the very broken grade line and the high rates of grade would apparently tend to create a somewhat irregular demand, as there would be nothing to prevent a combination frequently occurring where all the cars, or at least a majority of them, would be going up hill at once, and then within a few minutes be all going down hill. In the case of power-houses supplying power only for the operation of the cars on a railroad of this kind, this might be a serious consideration. The power-houses on this line, however, supply power for both manufacturing purposes and electric lighting, for which purposes the load is probably fairly even, and is a large proportion of the total output. The variation in the load from the operation of the cars, therefore, probably does not affect them greatly. The great economy of a uniform load at a power-station is strikingly exemplified by the contract recently made by the Com-

Mr. Lavis. monwealth Edison Company, of Chicago, for supplying power to the street railway companies of that city.* The very low rate of 0.4 cent per kw-hr. was made, provided the demand was uniform, any variation from this uniform demand being penalized according to a certain schedule mutually agreed upon. Of course, a street railway operating in any large city will have high peak loads during the rush hours, and therefore the demand cannot be regular; this instance. however, being so recent, and the contract so important, serves to emphasize the desirability of uniform loads at the power-houses.

It is well to bear this in mind and not lose sight of the fact that the electrical operation of railways has its limitations, although many liberties may be taken with the grades and alignment that would not be permissible on steam railroads.

There is another question which it has seemed to the speaker might naturally occur to some of the readers of the paper, and that is: Before deciding on the construction of these lines at all, was any consideration given to the possible effect on the new line, should the steam railroad, with its truly easy grades and curves, decide to electrify its line and run short trains at frequent intervals, or even equip its line with one of the many types of motor cars now apparently in successful operation on many steam railroad lines, where frequent service for a comparatively small number of passengers at a time is demanded? Probably the steam railroad would be at a disadvantage by its inability to reach the business centers of the various towns, and also to leave every passenger nearly at his own front door. It is quite probable, also, that such a service as that necessary to compete successfully with the electric lines might interfere too much with the through business of the road, both passenger and freight.

In constructing an interurban railroad across country, some years ago, where 60-ft. rails were used, the speaker had some trouble from the excessive expansion and contraction in such comparatively long lengths, until the ballast was filled in as close as possible to the top of the rails and on both sides of them. If the temperature changes are at all large, and the joints are left open sufficiently to prevent the track from kinking in hot weather, there is likely to be excessive pounding of the ends of the rails at the joints, due to the wide spacing. It would be interesting to know whether any such trouble has been found on this road; and, if so, what means were taken to overcome it. The standard cross-section of the track, shown in Fig. 3, seems to show that no ballast was placed above the tops of the ties.

It is quite common practice now, in laying street railway tracks through paved streets, to lay a fairly substantial concrete foundation both under and around the ties, and to such a height over them as will just allow for the type of paving to be used, with the necessary

* *Engineering News*, December 10th, 1908.

sand cushion under it. The speaker had occasion to note only a short time ago the installation of this type of construction for a street railway track operated by a large public service corporation, on a double-track line, where the cars were operated on 20 min. headway, with little if any special excursion business, so that the traffic was probably not greater than on the line described by Mr. Francis. No mention of such a type of track construction is made in the paper. If it was not used through the paved streets in the cities, was it on account of the necessity of curtailing expenses, or because it was not considered economical, in view of the expected traffic on the road? The excessive wear and tear on the motors of electric cars due to poor track has led to the very general adoption of the very best type of track construction possible for street railroads almost everywhere. **Mr. Lavis.**

The width between centers of tracks, adopted on this road, namely, 9 ft. 8½ in., is the general practice on street railway work in cities, though many interurban roads, where high speed is used, have adopted a wider spacing through the open country. On the road recently completed between Baltimore and Washington the tracks were laid 11 ft. on centers, and 80-lb. rails in 33-ft. lengths were used. The fact that the bridges on the Ohio road were designed to allow the tracks to be spaced 12 ft. on centers allows the inference that the engineers had in view the possible necessity of wider spacing of the tracks in the future.

There is another point which the speaker believes is not made quite clear in the paper, and that is the reason for building a double-track road. The only information as to the amount of traffic is the statement that cars are operated on 30 min. headway for ordinary traffic, and that there is a considerable excursion business. On roads with cars operating normally on 30 min. headway, it is generally quite satisfactory to build a single-track road with sufficient sidings to provide for operation on half that headway, the speaker having in mind a road in New England which handles quite successfully a very large summer excursion business on this basis.

The construction of steam railroads is now carried out along lines which have become fairly well standardized, and the type of construction necessary to meet certain conditions is in a general way well known to engineers at all familiar with this branch of the profession. The construction of electric railroads, however, owing to the much greater flexibility of the application of power, is subject to much greater variations, and a most intimate co-relation of the work of the engineers responsible for the location of the line and the electrical engineers is necessary in order to produce the most economical results. The practice in the construction of this road, as described by Mr. Francis, is so radically different from steam railroad practice that the speaker believes that a fuller statement of the reasons which led to the final design, so to speak, of this road, would be most interesting, not

Mr. Lavis. only to the members of this Society, but to all engineers interested in work of this class.

Mr. Preston. GEORGE B. PRESTON, ESQ.—As having some bearing on the relation between the average and maximum loads on the power stations, it may be of interest to note that with the heavy holiday load of July 4th, 1908, the 1-hour maximum at the East Liverpool house was 2 000 kw., while the average load during the heaviest part of the day, from 7 to 11 P. M., was 1 800 kw. On an ordinary day the 24-hour average for this station is 850 kw.

When considering the question of peak loads, in connection with this system, it should be borne in mind that these two power-houses are furnishing all the commercial and city lighting, as well as carrying a considerable commercial motor load, for four cities having an aggregate population of 73 000.

Mr. Boucher has asked whether or not the possibility of using the third-rail was considered in connection with this railway system. It was not considered feasible to use the third-rail system because of the necessity of utilizing the local electric railways at Steubenville and East Liverpool which were already installed and equipped with the overhead trolley, and also from the fact that a considerable portion of the roadbed lies in public streets and highways.

Mr. Schreiber. J. MARTIN SCHREIBER, ASSOC. M. AM. SOC. C. E. (by letter).—Although no new engineering features, other than represented in modern practice, are brought out in the paper, it is an intelligent and comprehensive description of a very interesting electric railway.

It is gratifying, at least to those who have had experience in the maintenance of some of the old-time electric railways, to know that the capitalist is rapidly realizing that it even pays to spend some money on the permanent improvement of street railways. Indeed, the very near future will see almost all electric roads constructed with as much skill and forethought as is represented on steam roads. This will be necessary for economical, satisfactory, and profitable operation, and in order to compete with other carrying companies.

The physical difficulties encountered in building the track and roadway of the Ohio Valley lines were out of the ordinary, as the location was not favorable for the work, and the author seems to be justified in the assertion that the configuration of the country through which the road was built is such as practically to preclude the location and construction of a future competing line. No doubt this condition accounts for the variable and at some places severe grades of the track. However, since the road is double tracked, successful and satisfactory operation is facilitated.

The author states that 85-lb. rails, in 60-ft. lengths, and of the Am. Soc. C. E. section, were used throughout the construction and

reconstruction. The writer is of the opinion that rails in 60-ft. lengths are not altogether desirable on open railway work. It is true that there are fewer joints and more of a continuous track with a longer rail, but there is a disadvantage in handling them, and difficulty in maintaining proper alignment. Unless special expansion joints are used, 60-ft. rails will kink in summer and have open joints in winter. The 33-ft. rail seems to work to the best advantage for open track, and the 60 and 62-ft. lengths for city or paved track. Mr. Schreiber.

From one of the photographs it appears that the rail in paved city streets is also of the T-section. The advisability of using the T-rail in paved streets and adopting it as a standard in all street railway work has been strongly advocated by many railroad men. The Way Committee of the American Street and Interurban Engineering Association will make this subject its principal work during 1909, and it is hoped that a definite recommendation will be offered at its next annual convention.

A number of authorities, including the engineers of cities, are co-operating with the street railways in permitting the installation of T-rails in the streets. Several of the up-State New York cities, including Utica and Syracuse, laid T-rails during 1908, and they are reported as very satisfactory; and such cities as Minneapolis, with T-rails and brick paving, claim to have some of the finest street-railway track and roadway.

The great advantage of the T-rail, over any other section, from an operating standpoint, and especially with the increasing wheel loads, is too well known to explain at length, and the question is often only in regard to permission from the authorities for its installation, as they generally object on the ground that the rail does not allow proper paving facilities, and that the paving wears too rapidly along the gauge line of the rail.

It is indeed unfortunate that in the early installation of T-rails in city streets, the paving, especially adjoining the rail, was not properly executed. This placed the T-rail in disrepute for municipal work, so that it is not uncommon for the engineer of a small borough to demand the installation of a Trilby rail, similar to that required in Philadelphia or New York. The writer has often found this to be the case where vehicular traffic was practically nil, and where the paving was macadam. Here the T-rail would not only be cheaper and a great deal more desirable by the railroad company than any other of the common girder sections, but would give better service and more satisfaction to the public at large. This paving about such a rail is as easily maintained as that adjoining the Trilby rail, and with the Trilby section the broken stone is continually getting into the groove and breaking wheel flanges, causing uncomfortable riding, disagreeable noises, and

Mr. Schreiber. frequent derailments, not to mention the trouble of maintaining the line and gauge.

On account of the great improvement made in T-rail track construction, it is the consensus of opinion of railway engineers that it is only a question of time when municipal authorities will co-operate with railway companies in the adoption of the T-rail, except in the very largest cities, where the streets are narrow, and traffic is very heavy. Just how far the T-rail may be used in densely populated cities, and with heavy wagon traffic and narrow streets, where trucks are compelled to follow the tracks, is difficult to anticipate; but, for ordinary city and interurban railways the T-rail should certainly displace the girder section.

The great difficulty of maintaining track and keeping it in line, and with proper joints, with the Trilby and tram sections, are objections which must be overcome; and there are railway managements which, in all probability, will direct their efforts in the future to satisfying the municipal authorities, by building better track, with substantial foundation, proper drainage, and more improved pavements to suit the T-rail, rather than attempt to operate with an unsatisfactory rail that will meet the present paving conditions.

Mr. Boucher. WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.—This paper is interesting in several particulars, and draws one's attention to the very extended development of cross-country electric roads competing with steam roads in Ohio and Indiana, many of which publish time-tables and schedules, run "limited" trains, and traverse distances from 50 to 100 miles on one division. As these roads are primarily to connect centers of population, they necessarily run through sparsely settled districts, and frequently on private right of way; the query then arises, why are so few roads operated by the "third-rail"?

In any consideration of the cost of installation, it is necessary to specify the kind of work wanted. Single-track, span-wire trolley construction may be had at a variety of costs, but, for similar operating conditions and equivalent current capacities, there is a marked difference in the cost of overhead and third-rail construction. Assuming, then, that first-class construction in both cases is called for; also similar operating and current conditions; a double-pole, single overhead trolley with feed wire; other material and labor necessary for erection may be had at a cost of from \$4 600 to \$5 000 per mile, depending considerably on the nature of the ground in which the poles are placed. An equivalent third-rail system, using an 80-lb. rail on extended ties and insulators, with splice-plates, bonds, and underground cable for road crossings, will cost, erected, about \$3 800 per mile of single track. The third-rail is unprotected. Board protection, similar to that installed in the New York Subway, will cost about \$2 000 per mile. The maintenance charges should be considerably less for the third-rail system over a period of years.

Noticing, then, this difference of $800 to $1 200 per mile in favor of the third-rail system, there must be some compelling reasons why the overhead trolley is used so much more frequently. Snow should not cause trouble to a third-rail if the interval between cars is no greater than 30 min., and during a storm any road must be prepared to run all night in order to keep the road open. Sleet, however, may cause trouble, but a scraper has been devised and is in use in New York which seems to overcome the difficulty. Of course, in the case of roads operating through country and towns, the frequent shifting of shoe and trolley would prove a nuisance, especially if a double-throw switch must also be operated, and without the latter the shoe is dangerous to passengers who congregate around the steps and near the trucks while waiting to board the cars. In the case of long-distance roads, however, with expanses of country and few towns, and running alongside a platform at stations, the third-rail would seem to be superior. If the road on its private right of way is properly fenced, and cattle guards are used at crossings, but little trouble will be caused by cattle and horses. Mr. Boucher.

The following roads are now operating with the third-rail: Albany and Hudson; Wilkes-Barre and Hazleton; Lackawanna and Wyoming Valley; Aurora, Elgin and Chicago; Scioto Valley, Ohio.

George B. Francis, M. Am. Soc. C. E. (by letter).—Referring to the discussion by Mr. Lavis: There were a few grade crossings created on the private right of way portion of the railways, but these could not be avoided with reasonable economy, and their creation was not prohibited by law or the public authorities. Mr. Francis.

The railways described are of the mixed type, comprising electric city street railway and interurban railway, for passenger service only. These conditions oblige and warrant radical departures in lines and grades from steam railroad practice, even on the private right of way portions. These portions, however, were located so as to conform substantially to steam railroad practice, taking into consideration character of power, equipment, and service. Speeds on lines of this character, operated with the type of wheel flanges required in usual street railway service, are necessarily less than would be the case with master car builder's steam railroad wheel flanges.

The steep grades indicated on the profile occur in places where the line is in public streets, and are governed thereby. Private right of way location was out of the question in such places, for many reasons, mainly because it was desirable to occupy certain main streets in best serving the public and increasing the revenue.

Consideration was given to the possibility of the parallel steam railroad improving its passenger facilities by electrification or otherwise, but, for various reasons, the probability of their so doing was thought to be too remote to govern the construction or non-construction of the new line; the main reason being the interference of local

Mr. Francis. passenger service with the requirements for freight service, in this particular locality, on the steam railroad lines.

No trouble has been experienced from expansion of the 60-ft. rail length adopted on this road. Such troubles, however, have occurred in situations of peculiar character elsewhere.

No concrete foundations were used for either the track or street paving involved. The local street vehicular traffic did not seem to require the placing of such foundations under the paving, the brick paving generally used in this locality, laid on a sand or gravel ballast, appearing to retain a good surface after many years' use.

The road was double tracked for several reasons, viz., safety in operation, adherence to schedule, adaptability to variable schedules (such as are demanded by holidays or changing conditions of service), and because such was the strong desire of the management controlling the properties.

By referring to *Engineering News* of December 10th, 1908, it will be seen that Mr. Lavis' statement: "the contract recently made by the Commonwealth Edison Company, of Chicago, for supplying power to the street railway companies of that city," providing for a rate of 0.4 cent per kw-hr., conveys a wrong impression (probably unintentional on the part of Mr. Lavis), and that an additional payment of $15 per annum per kw. of maximum demand is provided for in that contract; and this brings the cost to about 0.74 cent per kw-hr. The steady load making this figure possible is not the railway load, but the general lighting load of the Edison Company, which equalizes the railway peaks.

Mr. Preston, in his discussion, appears to have covered the point about the third-rail raised in Mr. Boucher's discussion.

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NICKEL STEEL FOR BRIDGES.*

By J. A. L. Waddell, M. Am. Soc. C. E.

With Discussion by Messrs. Charles Evan Fowler, M. F. Brown, H. P. Bell, L. J. Le Conte, W. K. Hatt, John C. Ostrup, T. Claxton Fidler, Robert E. Johnston, Albert Lucius, G. Lindenthal, Henry S. Prichard, Henry Le Chatelier, A. Ross, L. Dumas, Victor Prittie Perry, W. H. Warren, William R. Webster, William H. Breithaupt, E. A. Stone, C. Codron, W. W. K. Sparrow, B. J. Lambert, William Marriott, Henry Rohwer, Samuel Tobias Wagner, A. W. Carpenter, Leon S. Moisseiff, James C. Hallsted, F. Arnodin, Wilson Worsdell, William F. Pettigrew, and J. A. L. Waddell.

In November, 1903, the writer began a series of experiments, upon the comparative values of nickel steel and carbon steel for bridge building, for the purpose of preparing from them an exhaustive economic study of the subject of "Nickel Steel for Bridges." These experiments extended over three years.

The first step was to collect and read all the literature upon nickel steel bearing upon the subject, and, as this was rather meager, the task was soon completed. Although much was learned in this way concerning nickel steel in general, little was ascertained about the composition of the alloy most suitable for bridge building.

* This paper, as originally prepared, was much longer and more elaborate than as herein given; but, for the sake of economy of space, its dimensions have been reduced in every possible way that did not injure its continuity or militate materially against its ultimate object. The original paper with its six appendices is to be found in the Library of the Society, where those who desire any of the detailed information it contains and this paper does not, may refer to it. As it gives every step in the development of the investigation, and shows how the writer's ideas and intentions were modified from time to time as the inquiry proceeded, it may prove of interest to those who are contemplating making important engineering investigations.

At the outset, the writer retained the Osborn Engineering Company, of Cleveland, Ohio, to conduct the experiments, under his immediate direction and supervision; and F. C. Osborn, M. Am. Soc. C. E., the President of that company, at first gave his personal attention to the work, but, as soon as it was laid out, George C. Saunders, Assoc. M. Am. Soc. C. E., was placed in charge for the company.

In order to ascertain what was known of nickel steel suitable for bridge building which had not been recorded in print, the writer, in company with Mr. Osborn, made trips to Pittsburg and Pencoyd to consult with various officers and employees of the Carnegie Steel Company and the American Bridge Company. The result of these visits was very satisfactory; for much valuable information was thus obtained, and some material from two old melts of nickel steel was secured for the preliminary testing.

At the initiation of the work, a lay-out of tests covering the entire proposed investigation was prepared, and this was followed quite closely; nevertheless, as the testing proceeded, it appeared advisable to drop certain tests and to add others. One large group had to be abandoned on account of expense, as it involved the provision of another large melt of nickel steel; but from a number of tests on some other nickel steels the necessary deductions could be made.

The initial lay-out of the work involved the following:

1st.—The determination of the proportions of nickel and carbon that will provide a steel of the highest ultimate strength and elastic limit, consistent with perfectly satisfactory manipulation of the metal in the shops, and with absolute safety in the operation of structures built of it;

2d.—The making of a complete series of tests of the nickel steel or steels adopted as standard, and a corresponding series of tests of the ordinary carbon steel used in the manufacture of bridges, so as to determine the values of all intensities of working stresses for nickel steel in bridges;

3d.—The preparation of a thorough and minutely detailed set of specifications governing the designing of railroad and highway bridges in nickel steel;

4th.—The preparation of diagrams of weights of metal per linear foot of span, for all ordinary types of both single-track and double-track railway bridges, and for all spans, from that of the shortest plate girder up to that of the longest practicable cantilever, all designed

for modern live loads and according to standard specifications, these diagrams to establish the weight curves for structures entirely of carbon steel, those entirely of nickel steel, and those of mixed nickel and carbon steels, in which the latter material is used for all parts where there would be no advantage in having the stronger metal;

5th.—The preparation of a set of diagrams giving in comparison the total actual costs per linear foot of span for metal erected in nickel-steel and carbon-steel structures, and in mixed-steel and carbon-steel structures, for all the bridges of which the weights were diagrammed, and for all possible pound prices for carbon steel erected (varying by ½ cent), and for all possible variations in pound prices of manufactured nickel steel and manufactured carbon steel delivered at the bridge site. These diagrams would enable anyone to perceive at a glance the economics of nickel steel for all standard types of railroad bridges and for all possible conditions of the metal market; and, moreover, they would be good for all time, provided the characteristics of the two metals remained constant.

The initial step in making the investigation was, of course, to determine the best percentage of nickel to use. At first thought, this appeared to be purely a commercial question, the idea being that the greater the percentage of nickel the greater the strength of the alloy, but, at the same time, the greater the cost per pound, and that, consequently, there must be some one percentage of nickel which is more economical than any other. This is true enough, but the enquiries made at Pittsburg indicated that when the percentage of nickel exceeds a certain amount (then not determined definitely) the material becomes too refractory for shop manipulation or too expensive in manufacturing.

At the outset the writer had expected that it would be necessary to make a number of small special melts of nickel steel containing varying proportions of nickel, carbon, and possibly manganese, in order to settle the question of the best composition for the alloy; but the enquiries referred to and the preliminary tests on the specimens of nickel steel secured enabled him to determine this point without that necessity. Curiously enough, the superior limit of nickel for workability in bridge shops varies but little, if any, from the economic amount based on present prices of steel and nickel. For plates and shapes, which form the principal part of bridge superstructures, any materially greater percentage of nickel than 3½ renders the alloy too

refractory for the various shop manipulations to which it would be subjected in manufacturing it into bridges, although superstructures built of steel still higher in nickel would be perfectly safe and satisfactory for operation.

The writer learned also, both by enquiries and by the preliminary tests, the best percentages of carbon to adopt for nickel steels of the various kinds required in bridge building; and he is of the opinion that any elaborate investigation of this question that might be made would not modify his conclusions materially. These best percentages of carbon will be indicated later, in connection with the discussion of the tests. For the present it will suffice to make the general statement that the greater the percentage of nickel in the alloy the greater the permissible amount of carbon, as far as safety against brittleness is concerned; and that, as the addition of carbon to the steel costs nothing and adds materially to both the ultimate strength and the elastic limit of the metal, it is economical to use as much of it as a proper consideration of workability in the shop and safety in operation of structure will permit.

After making the enquiries at Pittsburg and Pencoyd, the writer, in consultation with Mr. Osborn, adopted the following maximum percentages of impurities in the nickel steel recommended for bridge work:

Phosphorus...............	0.03%
Sulphur..................	0.04%
Silicon..................	0.04%

At the same time it was decided to leave acid open-hearth steel out of consideration, and to assume that structural nickel steel will be manufactured exclusively by the basic open-hearth process.

As for the proper percentage of manganese, it was decided at first that this should be limited to about 0.6%; but, later, it was found advisable to increase this to 0.75% for plate-and-shape steel, and to 0.85% for eye-bar steel. Manganese adds to the strength of the alloy and costs comparatively little, but its use in excess renders the metal unduly hard.

The following is a list of the comparative tests of nickel steel and carbon steel which were made in the investigation:

A.—Specimen tests for tension, showing the elastic limit, ultimate strength, elongation, and reduction of area for the several kinds of steel required,

B.—Bending tests on specimens, both plain and nicked, to determine the angle of bend and the character of fracture,

C.—Bending on pins, to ascertain the extreme fiber stress for both elastic limit and ultimate strength,

D.—Bearing on pins and rivets, to determine the proper intensities of working stresses for bearing,

E.—Shear on rivets, to find the correct intensity of working stress for shearing,

F.—Tests for torsion, to ascertain the safe limit of extreme fiber stress on shafts of operating machinery,

G.—Impact tests, to learn the relative resiliences of nickel steel and carbon steel,

H.—Combined impact and tension tests, to show the comparative resistances of the two metals for this combination of conditions,

I.—Tests of full-sized eye-bars,

J.—Tests of full-sized columns, both short and long, to aid in the preparation of proper formulas for nickel-steel struts,

K.—Hammering-flat tests,

L.—Drifting tests,

M.—Close-punching tests,

N.—Planing tests,

O.—Drilling tests,

P.—Chipping tests,

Q.—Filing tests,

R.—Specific gravity tests,

S.—Coefficient of elasticity tests,

T.—Corrosion tests, to determine the comparative resistances of the two metals to the attacks of acid, damp salt, smoke, and wet cinders.

Analyses of the metal from the two old melts of nickel steel obtained for testing, and the medium-carbon steel used for comparison, gave the results shown in Table 1.

The percentages in Table 1 were obtained from drillings; but analyses of samples taken from the ladle gave, for the low-nickel steel, manganese, 0.68, carbon, 0.35, and nickel, 3.20; and for the high-nickel steel, manganese, 0.65, carbon, 0.42, and nickel, 4.22.

TABLE 1.—RESULTS OF ANALYSES OF STEELS.

Character of alloy.	Percentages of Ingredients.				
	Sulphur.	Phosphorus.	Manganese.	Carbon.	Nickel.
Low-nickel steel...........	0.015	0.011	0.65	0.39	3.21
High-nickel steel...........	0.014	0.009	0.67	0.463	4.25
Medium-carbon steel......	0.021	0.011	0.46	0.275	....

The following is a condensed record of all the preliminary tests, which are given in full in Appendix C.*

TENSILE TESTS.

An average of eleven tensile tests of the low-nickel steel gave 61 300 lb. per sq. in. for the elastic limit and 99 300 lb. per sq. in. for the ultimate strength. An average of fourteen tests of the high-nickel steel gave 71 600 lb. per sq. in. for the elastic limit and 114 000 lb. per sq. in. for the ultimate strength.

BENDING TESTS.

The bends were made on unannealed specimens, 2 in. wide and 10.5 in. long, with planed edges. The warm bends were made at ordinary shop temperatures, ranging from 55 to 70°, and the cold bends after the specimens had been immersed for 2 hours in a mixture of salt and ice. Four warm and four cold bends were made on each thickness of plates and angles of each of the three kinds of steel. The effect of the low temperature was quite marked. It seemed to be greater in the high- than in the low-nickel steel; but the tests were too few in number to warrant stating this conclusively.

All the low-nickel steel at the ordinary shop temperature was bent 180° about two diameters; the high-nickel steel was bent 120° about three diameters. The cracks developed much more suddenly in the high-nickel steel and in the cold bends than in the low-nickel steel and in the warm bends, and were greater in magnitude. The bends of the low-nickel steel specimens were better than those of the high-nickel steel, but were not as good as those of the carbon-steel specimens.

IMPACT TESTS.

There were four different series of impact tests. The first was made on specimens 21 in. long, ¾ in. wide and 2 in. deep, reduced at

* This appendix forms a part of the original paper, which may be found in the Society's Library. It consists of an itemized report of all the preliminary tests by Mr. Saunders.

mid-length by two notches, each ½ in. wide and ½ in. deep, leaving an effective depth of only 1 in. Five pieces of each kind of steel were broken, the specimens being reversed after each blow.

The apparatus, Fig. 1, was a rather crude affair, and was operated with much labor. The weight was a 50-lb. box, hoisted by a cord running over a small drum. It was raised and lowered by a worm gear turned by a small crank. Additional weights, in units of 50 lb., were added by placing castings on top of the box. The weight was released by disengaging a catch.

The test pieces were supported on rounds, 12 in. between centers, and the load was transmitted through a tongue rounded at the bottom so as to bring the effect of the blow midway between the supports.

Contrary to what had been gathered from the perusal of a number of papers on nickel steel, it was found by these experiments that the resilience of the nickel-steel alloys was considerably less than that of the carbon steel. All three steels were submitted to the same impact tests, the weight at first being 100 lb. and the drop increasing gradually from 6 to 24 in.; then, after twelve blows, the weight was increased to 150 lb. Calling the resulting average resilience of the carbon steel 100, that for low-nickel steel was 87 and that for high-nickel steel was 73.

This result might have been predicted, for turning over the specimens after each blow caused the total amount of abuse given to the metal to be a measure of its toughness and not of its resilience.

The second series of impact tests was similar to the first, except that the specimens were not reversed. The result gave the following comparison of resiliences:

Carbon steel	100
Low-nickel steel	117
High-nickel steel	109

Because of the contradictory results of the two series of impact tests, it was decided to make a third, in order to determine the relative resistances of the three steels to a sudden blow.

To make the tests, the apparatus shown in Fig. 2 was constructed. The hammer was forged round, with an eye-bolt in the top, and weighed 205 lb. It was raised by a traveling overhead crane. The specimens were similar to those used in the first series of tests, except that their length was reduced to 18 in., and there were ten of each of the three kinds of steel.

No piece was struck more than one blow, and the height of drop first selected was such as would positively break the specimen. With the succeeding diminishing heights, an effort was made to approximate to that which would just break the piece.

The comparative resiliences found by this series of tests were:

Carbon steel..................	100
Low-nickel steel...............	78
High-nickel steel..............	71

Thinking that the unsatisfactory showing of nickel steel under impact, as compared with carbon steel, and the diametrically divergent results of these impact tests, as compared with the results recorded by other experimenters, might be due to the notchings of the specimens, a fourth series, similar to the third, was made, but using plain bars of a section corresponding to that of the notched portion of the specimens previously tested, or $\frac{3}{4}$ in. in width and 1 in. in depth. It proved necessary to use a 500-lb. hammer, as the 200-lb. hammer used previously was entirely insufficient to break the specimens or to bend them materially. As it was, the tests were imperfect and inconclusive, for the high-nickel steel specimens broke under a drop of about 9.5 ft., those of low-nickel steel bent through an angle of from 120 to 130° under a drop of 12 or 13 ft. (only one piece breaking, and the others showing very slight indications of fracture), and those of carbon-steel bent under a drop of about 13 ft. without showing any signs of fracture. As the limits of the apparatus were reached, no attempt was made to carry the experiments any further.

Although the result was imperfect, the test is quite satisfactory in that it shows clearly the great amount of abuse that low-nickel steel will stand, and that, in this particular, it is quite suitable for bridge building; although, of course, it is not as tough and ductile as carbon steel. Had the metal been at all brittle, the specimens would have been shattered instantly by so large a weight falling from so great a height, instead of first bending through such large angles.

Up to the present, the general consensus of opinion has been that nickel steel has a higher resilience than carbon steel, and it was a matter of surprise to the writer that three of his four series of impact experiments indicated the contrary. On this account a thorough discussion of this feature of the paper is desirable. It may be concluded

that at present the comparative resiliences of nickel steel and carbon steel are still an open question; and it is hoped that the discussion will throw sufficient light upon the matter to settle finally the existing uncertainty.

It must not be forgotten that, in comparing the resiliences of nickel and carbon steels, if the strengths of the two metals be equal, or if their carbon contents be equal, nickel steel will give better results than carbon steel; and it is only when comparing nickel steel high in carbon with the ordinary low-carbon steel used in bridgework that the showing is ever unfavorable to the alloyed metal.

Combined Impact and Tension Tests.

The writer made an attempt to ascertain the effect of dropping a weight upon an eye-bar under tension, but the test was a failure. The weight (200 lb.) and the drop (4 ft.) were both too small. Although the bar was finally broken, it was impracticable to determine whether the ultimate strength was affected by the impact, as the piece developed a pretty high resistance for medium-carbon steel.

To obtain any results of value, it would have been necessary to set up a small pile-driver over the bar and drop a much larger weight from a greater height and with far greater frequency. After this experience it was concluded that "the game was not worth the candle," consequently the intention of carrying this test any further was abandoned.

Later, the writer retained W. K. Hatt, Assoc. M. Am. Soc. C. E., of Purdue University, to make, among other tests, some experiments on combined tension and impact, and combined bending and impact. His full report is given in Appendix E.* The following extracts from it will show briefly his findings:

"The impact tension tests were made on the machine in the Purdue Laboratory used by the writer in previous tests, and described in the *Proceedings* of the American Society for Testing Materials, Vol. 4, 1904. The hammer used weighed 810 lb. This was allowed to drop a distance, h, in such a way that the energy of the blow was absorbed by the test piece. If the piece broke under the first blow, a record, taken on a revolving drum, made it possible to compute the energy left in the hammer after rupture. In nearly every case the piece was ruptured by one blow.

* On file in the Library of the Society, but not reproduced in this paper.

"The impact flexure tests were made upon an electrically-operated impact machine. The hammer used weighed 55 lb. The 3 600-lb. base was supported by a 6-ft. concrete foundation resting upon undisturbed gravel. The bars were tested on a span of 18 in., with the machined faces on the top and bottom. The loads were applied by allowing the hammer to drop upon the center of the beam from varying heights, without turning the bar over between blows. The corresponding deflections were indicated by a recording pencil on the slowly revolving drum. The routine of each test was as follows:

"With the hammer just touching the bar, a zero line was marked on the drum. The hammer was then allowed to fall from a height of ¼ in. on the bar, the deflection being marked on the slowly revolving drum. These operations were repeated, increasing the height of drop ¼ in. each time, until the elastic limit was well passed. When the elastic limit was reached, the material showed signs of set. This was marked on the drum by the recording pencil by turning the drum slightly after the vibrations of the bar had ceased. A typical drum record is shown on Fig. 3. None of the specimens, except the nicked bars, were broken in flexure.

"Two of the nicked specimens, * * * one of carbon steel and one of nickel steel, were tested in a different manner from that outlined above. The hammer was allowed to fall from a height of 18 in., then from a height of 24 in., and then from a height sufficient to cause rupture. The drum records for these tests were similar to those obtained in the impact tension tests."

Professor Hatt's results of impact tests are given in Table 2, which forms a part of his report.

TABLE 2.—RESULTS OF IMPACT TESTS.

	Carbon.				Nickel (3.5%).			
	No.	Average.	Maximum.	Minimum.	No.	Average.	Maximum.	Minimum.
Impact Tension.								
Elongation	5	31.5	32.0	31.0	4	16.5	19	13
Contraction	5	58.9	60.8	57.0	4	49.7	54	45
Rupture-work	5	1 736	1 910	1 540	3	2 198	2 300	1 960
Impact Flexure. Plain bar.								
Deflection at blow from 3 in.	8	0.18 in.	0.21 in.	0.16 in.	5	0.09 in.	0.11 in.	0.09 in.
Deflection at elastic limit	8	0.21 "	0.23 "	0.16 "	5	0.26 "	0.30 "	0.22 "
Height of drop at elastic limit	8	3.70 "	3.50 "	3.00 "	5	8.10 "	9.00 "	7.25 "
Impact Flexure. Nicked bar.								
Deflection at blow from 3 in.	3	0.20 in.	0.22 in.	0.17 in.	3	0.12 in.	0.14 in.	0.12 in.
Deflection at elastic limit	3	0.21 "	0.25 "	0.17 "	3	0.23 "	0.24 "	0.21 "
Deflection at rupture	3	1.23 "	1.40 "	0.9 "	3	0.62 "	0.70 "	0.57 "
Height of drop at elastic limit	3	3.30 "	3.37 "	3.25 "	3	6.10 "	6.25 "	6.00 "
Height of drop at rupture	3	11.10 "	12.75 "	9.50 "	3	15.60 "	17.00 "	14.5 "

Table 2 shows that, in the impact tension tests, while the elongation of carbon steel was nearly twice as great as that of nickel steel and the contraction was more than 20% greater, nickel steel had a resilience 26% higher than carbon steel—a result almost entirely at variance with the writer's findings.

Table 2 also shows, in the impact flexure on plain bars, an increase in resilience of nickel steel over carbon steel equal to 120% at the elastic limit, and in the case of nicked bars an increase of 85% at the elastic limit and 40% at the point of rupture.

Hammering-Flat Tests.

The hammering-flat tests were quite conclusive, and proved that both the nickel steels compare very favorably with carbon steel in their ability to permit of being hammered flat.

Drifting Tests.

The drifting tests, as anticipated, show that nickel steels will not stand quite as much drifting as medium-carbon steel, nevertheless, both grades would comply with the drifting requirements for high-carbon steel given in standard bridge specifications.

Close-Punching Tests.

The experiments show that both the nickel steels withstood close-punching quite as well as the carbon steel, in fact, in one respect better, because the flow of metal was markedly less in the stronger steels, and the holes were smoother and more regular.

Shop-Tooling Tests.

The shop-tooling tests included planing, drilling, chipping, and filing. They were quite satisfactory, and gave the best comparative results of all the tests made. They showed conclusively that high-nickel steel is unfit for bridge building (except for eye-bars, pins, and rollers), not because of any inherent defect in the alloy, but because of its severity on the tools. To manipulate such metal in the shops, all machinery and tools would have to be built upon a more substantial basis than for carbon-steel work, and a stronger metal for punches and drills would have to be adopted.

As for low-nickel steel, the tests showed that this alloy is all right for shop manipulation, but the time required to do a certain amount of work upon it is greater than that required to do the corresponding work upon carbon steel.

Corrosion Tests.

The corrosion tests included immersion in a weak solution of sulphuric acid, in damp salt, in locomotive fumes, and in wet cinders. In order to avoid the possibility of accident, the writer decided to have these tests made in duplicate—in his own office, at Kansas City, and in the office of the Osborn Engineering Company, at Cleveland. Owing to failure to ensure in advance that the methods of testing would be exactly alike in detail, the results are not truly comparable.

All the plates tested were originally 6 by 6 by ½ in., and weighed about 5 lb. each. Three plates (one for each kind of steel) were used for each test, or twelve in all for each set of experiments.

Acid Test.—Mr. Osborn used a 1% solution of sulphuric acid, with a new bath every two weeks, and he did not clean the specimens; while the writer used a 2% solution, changed the bath every week, and scraped the specimens each time the bath was changed. The results of these tests are recorded on Fig. 4. From the writer's record it will be seen that in 160 days 94% of the high-nickel steel was dissolved, thus ending the experiment, and that in the same time the low-nickel steel lost 89% and the carbon steel 52 per cent. Mr. Osborn found in 890 days the following losses of weight: 13.2, 12.9, and 11.7%, respectively.

Both investigators were surprised by the results of this test, because they had been told by one of the Carnegie Steel Company's superintendents that the pickle test would show up very favorably for nickel steel, as he knew by having made the experiment.

This test, however, proves nothing against the use of nickel steel for bridges, because those structures are never exposed to such a condition or to any condition even approximating to it in severity, and because the other three corrosion tests, which agree in character more closely with the actual exposures of bridge metal, indicate a decided superiority of nickel steel over carbon steel.

Salt Tests.—As shown by Fig. 5, the writer's salt test extended over 522 days, at the end of which time the carbon steel had lost

2.65% of its weight, the low-nickel steel 0.72%, and the high-nickel steel 0.63%, showing that nickel steel resists the attacks of salt four times as well as carbon steel. Whether the two steels when immersed in sea water would offer the same comparative resistances to corrosion is hard to say, but it seems probable that they would. If so, this would be a strong point in favor of using nickel steel for ocean piers and for bridges located near salt water.

Mr. Osborn's results for the salt test were: In 840 days the carbon steel had lost 3.3%, the low-nickel steel 3.1%, and the high-nickel steel 3.2 per cent. The curves at 880 days cross each other, but this is probably due to some error in the records, since for 800 days the three curves are substantially parallel.

The great variation in the two sets of experiments must be due to the fact that the writer's plates were kept continuously in the solution and Mr. Osborn's were not. While this difference in treatment might account for the variation in the total amount of corrosion for any one kind of plate, it does not explain why the comparative resistances of the three kinds of steel should differ so greatly in the two sets of experiments.

Smoke Tests.—As shown by Fig. 6, the smoke test was continued by the writer for 250 days, when the plates were lost by the renewal of the floor system of the bridge beneath which they were suspended. His test at the end of that time showed a loss of 6.25% of its weight by the carbon steel, 4.5% by the low-nickel steel, and 4.4% by the high-nickel steel, indicating that nickel steel resists locomotive fumes about 40% better than carbon steel.

Mr. Osborn found the following results: In 310 days the carbon steel lost 0.18%, the low-nickel steel 0.12%, and the high-nickel steel 0.04 per cent.

The great difference in the results of the two tests was certainly caused by Mr. Osborn's failure to clean the plates from time to time, for the coat of soot deposited on the metal tended to preserve it from further attacks of the fumes.

Cinder Tests.—As shown by Fig. 7, the cinder test was continued by the writer for 520 days, at the end of which time the carbon steel had lost 3.45% of its weight, the low-nickel steel 2.5%, and the high-nickel steel 2.15 per cent. This is quite a favorable showing for the nickel steel.

Mr. Osborn's cinder tests gave the following results: In 890 days the carbon steel had lost 16.7%, the low-nickel steel 14.5%, and the high-nickel steel 14.1 per cent. Up to 740 days, however, the high-nickel steel corroded a little more than the low-nickel steel. The greater amounts of corrosion in the Osborn test must have been due to the nature of the cinders used.

From the various corrosion tests made by both experimenters, it may be concluded that nickel steel resists decidedly better than carbon steel the attacks of all substances which ordinarily tend to injure the metal-work of bridges.

Rivet Steel.

While the various preliminary tests were being made, a thorough investigation of the qualities of a suitable nickel steel for rivets was carried on, using specimens of old melts, in order to avoid the expense of making a special melt or melts for the purpose. While none of the five or six melts tested proved exactly suitable for rivets, enough was learned by the experiments to determine closely the proper proportions of the various constituents of rivet nickel steel. All the specimens tested were good enough for the purpose, except in one important particular, namely, the rivets were too difficult to cut out, the reason being that the carbon contents were too high.

In the writer's opinion, ideal rivet nickel steel should have about the following composition:

Nickel	3.5 per cent.
Carbon	0.12 to 0.18 per cent.
Manganese	0.55 to 0.65 per cent.
Phosphorus	0.03%, maximum.
Sulphur	0.04%, maximum.
Silicon	0.04%, maximum.

It is possible that a small economy might be effected by lowering the percentage of nickel and increasing that of carbon, and in time this may be done; but at present it does not seem worth while to make the nickel content less than that in plate-and-shape steel. The writer is of the opinion that by experimenting it could probably be shown that a rivet steel containing from 2 to 2.5% of nickel and from 20 to 25 points of carbon would answer every purpose; but this is only a surmise on his part. It is possible, though, that such a high percentage

of carbon would cause the rivets to burn too easily, but the writer's experiments gave no such indication.

In addition to the ordinary tests for tension and bending, a number of special tests on rivet nickel steel were made, among others, the following:

A.—Driving two rows of nickel-steel rivets and two of carbon-steel rivets so as to connect four thicknesses of ½ by 12-in. by 3 ft. 6-in. plates, reaming one row of holes for each kind, and noting how the nickel-steel rivets drove, as compared with the carbon-steel rivets:

The result was that no difference whatsoever could be noticed in the difficulty of driving or the excellence of the rivets, and this was found to be true throughout the entire series of experiments on nickel steel and carbon steel.

B.—Cutting off by hand the heads of most of the rivets mentioned in *A*, and driving them out of their holes, so as to obtain a proper comparison by noting the times required:

The results of this test were very erratic, but, in general, it was found that the higher in carbon the nickel steel the longer it took to cut off the heads. When a pneumatic chipper was used, it sometimes required twice as much time for nickel steel as for carbon steel.

In backing rivets out of reamed holes, no difference was noted in the resistances of the two kinds of rivets; but, in punched holes, the nickel-steel rivets came out in three-quarters of the time required for backing out the carbon-steel rivets. This result is due to the fact that the punched holes in nickel steel are much smoother than those in carbon steel. In this test (and in others also) it was learned that nickel-steel rivets should be used in nickel-steel plates and carbon-steel rivets in carbon-steel plates; for, when cutting out nickel-steel rivets from carbon-steel plates, the material around the rivet holes was injured because of the greater hardness of the rivet metal.

C.—Punching four ½ by 6 by 15-in. plates with two rows of badly-matched holes (but making the matching in the two rows correspond), assembling the plates, reaming the holes, and filling one row with nickel-steel rivets and the other with carbon-steel rivets, then planing off the sides of the plates up to the medial planes of the rivets, so as to show how well the holes were filled:

It resulted that there was but little difference in the flow in the two kinds of steel, what little there was being in favor of carbon steel.

D.—Flattening the ends of $\frac{7}{8}$-in. rivets, both cold and hot, and noting how the metal stands the abuse:

The cold nickel-steel rivets flattened to $\frac{7}{16}$ in. and then split, while the ends of the carbon-steel rivets flattened to $\frac{3}{16}$ in. before splitting. Undoubtedly, there would have been a better showing with nickel steel of ideal composition for rivets. The hot ends of both steels flattened out very thin without cracking.

E.—Flattening rivets, both cold and hot, end on:

In this test the cold carbon-steel rivets compressed 67% and the cold nickel-steel rivets 45 per cent. The latter required a much greater number of blows and received more damage. In the case of the hot rivets, both specimens flattened to a thickness of $\frac{3}{16}$ in. and to a diameter of $3\frac{1}{2}$ in. without showing signs of cracking.

F.—Testing rivets in double shear:

The result of the tests of rivets in double shear showed that the nickel steel was about 40% stronger than the carbon steel, the ultimate strength of rivet nickel steel in shear being about 59 000 lb. per sq. in., or about 75% of the tensile strength.

G.—Testing riveted joints:

The result of the test on riveted joints was to obtain an ultimate shear of 68 000 lb. per sq. in. on the nickel-steel rivets and 46 500 lb. per sq. in. on the carbon-steel rivets. The ratio of these two figures is about 1.46, which is a little greater than the ratio found for the shears on single rivets.

All things considered, the writer favors adopting rivet diameters for nickel-steelwork $\frac{1}{8}$ in. greater than those customary for carbon-steelwork. There are three good reasons for this, namely:

First.—The increase of strength of nickel-steel rivets over carbon-steel rivets is not commensurate with the increase of strength in the plate-and-shape metal. This necessitates either a greater proportionate number of rivets or a greater rivet diameter in nickel-steelwork.

Second.—The larger rivets keep hotter, and consequently fill the holes better.

Third.—The strength of the punches is increased about one-third, while the work of punching each hole is increased only about 13 per cent. This reduces the danger from broken punches, and makes the work in punching nickel steel probably not more hazardous than that in punching carbon steel.

Eye-Bars of High-Nickel Steel.

In order to determine approximately the kind of material for eye-bars the high-nickel steel would make, there were cut from the ¾-in. plates two strips, 5 in. wide, and these were forged into eye-bars, annealed, and tested.

The elastic limit of the first bar broken was lost. Each bar broke near the neck, at a point where the temperature stresses are the greatest, and this, in connection with the low percentage of elongation in the body of the bar, would seem to indicate insufficient annealing. The breaking of both bars in the same place, however, may have been simply a coincidence. The fractures were 100% silky, and the reductions of area were excellent.

The results of the two tests were as follows:

Number of test.	Elastic limit.	Ultimate intensity.	Percentage of elongation in 10 ft.	Reduction of area.	Fracture.
Bar 1..........	Lost...........	105 900	7.4	46.9%	Silky, ½ cup.
Bar 2..........	Uncertain	102 300	6.8	45.8%	Silky, ½ cup.

The unannealed specimen tests on the same plate gave an elastic limit of 71 100, an ultimate strength of 112 900, an elongation in 8 in. of 17.7%, and a reduction of area of 44.4 per cent.

These tests are valuable in that they prove that satisfactory eye-bars can be manufactured from steel containing 4½% of nickel and about 45 points of carbon.

Special Melts of Nickel Steel.

Two special 40-ton melts of plate-and-shape steel were made for the writer by the Carnegie Steel Company, under the supervision of Albert Ladd Colby, M. Am. Soc. C. E., to whose care and attention is due the fact that the metal obtained was greatly superior in uniformity to the low-nickel steel first tested. The composition of these special melts was determined by a number of independent chemists, with rather variable results, the general averages being as follows:

Nickel 3.65%, carbon 0.39%, manganese 0.77%, sulphur 0.025%, phosphorus 0.01%, and silicon 0.05 per cent.

The metal from these melts was rolled into plates and angles, some of which were used by the writer for testing. The remainder

was distributed among the principal bridge shops of the country to undergo various manipulations, in order to let the manufacturers judge for themselves of the ease or difficulty of manufacturing it into bridges. This distribution was done liberally, even lavishly, and it is hoped that those who received the metal have submitted it to many practical shop tests, and that they will give the results thereof to the engineering profession in the discussion of this paper.

The following is a list of the companies to whom material was thus sent:

Dominion Bridge Co., Ltd., Montreal, Que.;
Riter-Conley Mfg. Co., Pittsburg, Pa.;
The Canadian Bridge Co., Walkerville, Ont.;
The Hamilton Bridge Works Co., Ltd., Hamilton, Ont.;
Milliken Bros., New York City;
The King Bridge Co., Cleveland, Ohio;
McClintic-Marshall Construction Co., Pittsburg, Pa.;
Pennsylvania Steel Co., Steelton, Pa.;
Fore River Ship and Engine Co., Quincy, Mass.;
Newport News Shipbuilding and Dry Dock Co., Newport News, Va.;
William Cramp and Sons Ship and Engine Building Co., Philadelphia, Pa.

It was decided to postpone making the special melt of eye-bar steel until after the plate-and-shape steel had been tested. This was a wise precaution, because the testing of the latter might have caused some modification of the composition of the eye-bar steel; but the result, unfortunately, was disastrous, as the eye-bar steel was never manufactured. This failure reduces somewhat the value of this research, as compared with what was outlined when the experiments were first contemplated. The eye-bars for testing were manufactured from the plate-and-shape steel; and, although the results were very satisfactory, the writer is convinced that much stronger eye-bars can be readily manufactured by using steel higher in nickel, carbon, and manganese, and that they will be suitable in every way for use in bridges.

The special plate-and-shape nickel steel was tested very thoroughly and carefully, both in specimens and full-sized bridge members. Except in the case of eye-bars, for each test of nickel steel there was

made a corresponding test of medium-carbon bridge steel. The results are recorded at length in Mr. Saunders' report, which is given in the original paper,* and is reproduced in a slightly condensed form in Appendix A of this paper.

From it the writer has prepared the following summary and deductions:

TENSILE TESTS.

The average elastic limit is about 62 200 lb. per sq. in., and the minimum may safely be taken at 60 000 lb. for specimens cut from the edge; the average ultimate strength is about 107 300 lb. per sq. in., and the minimum may safely be taken at 105 000 lb. for edge specimens. For specimens taken from the edges of plates, the ultimate strength is about 3 000 lb. per sq. in. higher, and the elastic limit about 2 000 lb. per sq. in. higher than for specimens taken from the interior. This ruling does not appear to apply to eye-bar flats, for their interior metal is just as strong as, if not stronger than, the metal near the edges.

The elongation increases slightly with the thickness of the piece, and the reduction of area decreases as the speed of the testing machine increases.

The elastic ratio (that is, the ratio of elastic limit to ultimate strength) decreases as the thickness of the metal in plates or shapes increases; but the conditions affecting the testing of eye-bar metal were such as to prevent any reliable deduction being made on this point in relation to flats. This elastic ratio, in general, varies from 0.55 to 0.60, but occasionally there are cases that pass slightly beyond these limits.

Comparing the nickel steel and the carbon steel (which was of the kind specified as medium steel in "De Pontibus"), it was found that the average ratio of elastic limits was 1.94 and that of ultimate strengths was 1.79.

The minimum elongation in 8 in. was about 15% for nickel steel and 27% for carbon steel.

The minimum reduction of area was about 41% for nickel steel and about 55% for carbon steel.

It was found that thick plates and angles gave lower elastic limits and ultimate strengths than thin ones. This was already known to be

*On file in the Society's Library.

true for carbon steel, but the difference is so marked in nickel steel that any standard specification for the strength of that metal will have to take into account the thickness of the piece tested.

Difficulty was found in determining the elastic limit, or, more strictly speaking, the yield point; consequently, Mr. Saunders was compelled to make some arbitrary assumptions, as shown by his report.

It was noted during the tests that the nickel-steel angles were slightly inferior in strength to the nickel-steel plates, probably because of the smaller amount of work the angles received in the rolling, and possibly because of the bend. It has been recognized for years in carbon steel that plate specimens give slightly better results than shape specimens.

At the outset, the writer hoped to be able to obtain plate-and-shape steel which would have a minimum elastic limit of 64 000 lb., and that this might possibly be raised to 68 000 lb. by increasing the carbon content to 40 points. The experiments show an inferior limit of 60 000 lb., which might indicate that Mr. Colby's two melts of plate-and-shape steel did not come up to the writer's anticipations; but such is not the case, for, when that metal was tested by the ordinary mill methods, the average elastic limit was 65 600 lb. per sq. in. As the manufacturers' tests on the old melts were made by the usual mill methods, the writer based his judgment of possible results on that style of testing being adopted; but it was soon found that nickel steel should be tested much more slowly than has been customary for carbon steel, and the tests were conducted accordingly. In fact, the usual quick method of testing gives fictitiously great results for both the elastic limit and the ultimate strength of the latter metal. The elastic limit is especially affected. This feature of standard testing will be discussed further, when the subject of eye-bars is treated.

Tensile Tests on Punched, Reamed, and Punched-Riveted Specimens.

The results obtained by this series of tensile tests are as follows:

The punching alone raised the yield point in nickel steel from 2 to 11%, and in carbon steel from 5 to 24%, at the same time lowering the ultimate strength of the nickel steel from 9 to 11% and that of the carbon steel from 6 to 7 per cent.

This shows that the injurious effect of punching is a little more pronounced in nickel steel than in carbon steel.

In the sub-punched-and-reamed specimens, both the nickel steel and the carbon steel regained all their ultimate strength, and in fact a little more. This shows the truth of what has been contended by the writer for a long time concerning reaming, namely, that it is the only method which insures truly first-class shopwork and the full strength of the manipulated metal. It also shows that, while this is true for carbon steel, it is especially so for nickel steel.

The punched-riveted specimen tests indicate that driving the hot rivets into the punched holes increases still further the elastic limit and reduces the ultimate strength, thus furnishing another good reason for insisting upon sub-punching-and-reaming. The injurious effects of driving hot rivets in nickel steel and in carbon steel were about alike for the two metals.

Bending Tests.

The results of the bending tests on plain bars, when the bending is carefully and properly done, indicate that the plate-and-shape steel may be bent 180° around a mandrel having a diameter equal to twice the thickness of the test piece without showing any signs of cracking the metal.

Bending Tests on Punched, Reamed, and Punched-Riveted Specimens.

A study of Fig. 2, Plate XV, of the inspector's report in Appendix A proves very clearly the injurious effects of punching on both nickel steel and carbon steel, for, while the reamed specimens of nickel steel bent on the average about 90°, and those of carbon steel 180°, the punched specimens of nickel steel bent only about 50° and one of carbon steel only 84°; and the punched-riveted specimens of nickel steel averaged only 38°, and one of the carbon-steel specimens bent only 62 degrees.

This test is a strong confirmation of the necessity for sub-punching-and-reaming, and supports well the evidence of the tension tests on punched specimens.

Drifting Tests.

The result of the drifting tests proves that plate-and-shape nickel steel will withstand most of the ordinary drifting requirements for carbon-steel plates, but that, in the new specifications for nickel-steel

bridges, the required enlargement of the holes should be limited to 40 per cent.

Close-Punching Test.

This test proves that nickel steel can be punched as closely as carbon steel, and, in fact, that it withstands punching better, in that the edges of the piece are bulged less and that the punchings are smooth and regular while in carbon steel they are rough and irregular. Fig. 1, Plate XVII, might lead one to think that this conclusion is not true, as one specimen shows several broken partitions. These were caused by the nervousness of the operator, who, fearing the breaking of the punch, dodged his head behind a protection at the instant of punching, and thus in several cases got the holes too close together. Nickel steel will stand any specification yet written for the close-punching of carbon steel.

Hammering-Flat Test.

The result of the hammering-flat test shows that nickel steel will stand hammering almost as well as carbon steel.

Shop-Tooling Tests.

The shop-tooling tests were quite elaborate, and covered every important manipulation of metal in a bridge shop. It was found that nickel steel can be sawed with the usual saw and sheared with the usual shears. In punching, there was no special difficulty, but it is probable that more punches would have to be used for nickel steel than for carbon steel. If the diameters of all rivets in nickel steelwork are made ⅛ in. greater than those for carbon steelwork, as previously mentioned, there should be no greater number of punches broken, because, as before stated, the strength of the punch, which varies as the square of its diameter, would be increased about one-third, while the work of punching would be increased only about 13 per cent. Moreover, it is practicable to improve materially the quality of the metal in the punches.

In reaming, it was found that by the new process, termed "dry reaming," nickel steel can be reamed just as rapidly as carbon steel, but that the tools wear out more frequently. The old-fashioned method of reaming, in which oil or soapsuds are used for the lubricant, is not well adapted to nickel steel.

In drilling, the same remarks apply as in the case of reaming. The ordinary drills fail quickly in nickel steel, but "blue-chip" drills stand the work very well. The writer's drilling tests were made without a lubricant. Had one been used, it is likely that better results would have been obtained.

In planing nickel steel, it is not practicable to make cuts thicker than $\frac{1}{16}$ in.; while, in carbon steel, cuts of twice that thickness are usual. The effect on the tools is much greater for nickel steel than for carbon steel, and the amount of work that can be done on the former in a given time is much less than can be done on the latter—probably about one-half.

In pneumatic chipping, it takes about 50% more time to make a certain length of cut in nickel steel than in carbon steel, and the amount of metal removed is slightly less for nickel steel.

In hand chipping, a workman will cut in a given time only seven-tenths the length in nickel steel that he will in carbon steel, the thickness of the chips being the same.

Bearing-on-Pins Test.

It was found by the bearing-on-pins test that the plate-and-shape nickel steel is about 66% stronger than the carbon steel. In the case of bearing on rivets the excess of strength amounted to 83 per cent.

Bending-on-Pins Test.

Owing to the difficulty in testing round sections, square ones were substituted, and it was assumed that the comparative resistances for the two metals would be the same for both sections. It was found that nickel steel is about 85% stronger than carbon steel in resistance to bending.

Compression Tests of Struts.

The compression tests of struts were most interesting and important, especially as some engineers of high standing had surmised that there would be no economy in using nickel steel for long columns, because they anticipated that the two kinds of struts would deflect about the same for like total loads. It was found, however, that for short struts the nickel steel was 75% stronger than the carbon steel, and for struts of medium length 47% stronger. While these excesses of strength are not as great as those for tension, they are quite satisfactory.

Coefficient of Elasticity Test.

This portion of the investigation was done with extreme care, three tests being made for each kind of steel, in which the average values of E proved to be 30 312 000 for nickel steel and 29 420 000 for carbon steel. For convenience in calculating, it will be advisable to take the coefficient of elasticity for nickel steel at 30 000 000.

Tests of Eye-Bars and Eye-Bar Material.

Unfortunately, all the eye-bars tested had to be rolled of plate-and-shape steel. Bars 1 and 2 in. thick were used, but no comparison was made with carbon-steel eye-bars of the same sizes. Four 6-in., three 8-in., and one 16-in. bar were tested, and enough specimen tests of the metal were made to afford a proper average. The results of the tests are given in Table 3.

TABLE 3.—Results of Tests of Eye-Bars and Eye-Bar Material.

Six-Inch Bars.

Specimens.				Bars.	
Elastic limit	Unannealed	Average	60 600	....	
		Minimum	56 900	....	
	Annealed	Average	54 800	Average	56 400
		Minimum	52 700	Minimum	55 400
Ultimate	Unannealed	Average	103 100	....	
		Minimum	101 300	....	
	Annealed	Average	100 000	Average	101 000
		Minimum	98 700	Minimum	99 000

Eight-Inch Bars.

Elastic limit	Unannealed	Average	58 600	....	
		Minimum	54 700	....	
	Annealed	Average	52 500	Average	51 700
		Minimum	50 700	Minimum	48 300
Ultimate	Unannealed	Average	98 900	....	
		Minimum	93 800	....	
	Annealed	Average	95 200	Average	90 200
		Minimum	92 600	Minimum	88 900

Sixteen-Inch Bars.

Elastic limit	Unannealed	Average	59 100	....
		Minimum	58 500	....
	Annealed	Average	55 600	49 400
		Minimum	55 200	
Ultimate	Unannealed	Average	103 600	....
		Minimum	103 100	....
	Annealed	Average	103 200	96 900
		Minimum	102 700	

The following deductions have been made from the results given in Table 3:

The average effect of the annealing of specimens is to reduce their elastic limits about 9% and their ultimate strengths about 3 per cent.

The average elastic limits are for:

Unannealed specimens	59 400	lb. per sq. in.
Annealed specimens	54 300	" " " "
Full-sized bars	52 500	" " " "

The average ultimate strengths are for:

Unannealed specimens	101 900	lb. per sq. in.
Annealed specimens	99 500	" " " "
Full-sized bars	96 000	" " " "

The average loss in elastic limit between the unannealed specimens and full-sized bars is about 11%, and the corresponding loss in ultimate strength is about 6 per cent. It is more than likely that, in the future manufacture of nickel-steel eye-bars, these variations between the elastic limit and the ultimate strength of specimens and of full-sized bars will be lessened materially by a careful study of the process of annealing.

Cold-Pressed Threads Test.

An unsuccessful test on bolts of rivet nickel steel with cold-pressed threads was made and repeated. It seemed to be impracticable to manufacture such a bolt so as to have it break in the body before failing in the threaded portion; consequently, it was decided that nickel steel is not a proper material for the fabrication of bolts with cold-pressed threads. It is possible, though, that a nickel steel with a very low carbon content would work satisfactorily.

Tests for Specific Gravity.

Some very careful tests were made of the specific gravities of the two nickel steels and carbon steel, and it was found that the low-nickel steel was 0.017% lighter than carbon steel, and the high-nickel steel 0.043% lighter. As these variations are extremely small, it will be proper to assume the weight of nickel steel to be exactly the same as that of carbon steel, when estimating weights of metal in bridges.

Tests on Torsion.*

Professor W. K. Hatt made for the writer a number of experiments upon rods 1 in. in diameter and 36 in. long, the nickel steel being standard "plate-and-shape" steel, and the carbon steel that ordinarily used in bridgework. He found the modulus of elasticity in torsion to be 11 760 000 for carbon steel and 11 450 000 for nickel steel, the corresponding values of the modulus of rupture being 60 000 and 93 000, showing that low-nickel steel is about 55% stronger than carbon steel in its resistance to rupture, but practically equal to it in resisting distortion from twisting. Figs. 8 and 9 illustrate the results of Professor Hatt's torsion tests.

Specifications for Nickel-Steel Bridges.

From the results of the preceding investigations it is practicable to write specifications for nickel-steel bridges which will possess the same strength, rigidity, and general excellence of design as the best carbon-steel bridges that are being built to-day.

In preparing such specifications, the writer has adopted as a standard those given in "De Pontibus," modifying only those portions which are affected by the change of metal; and, in order to limit the length of this paper as much as possible, he presents here only the modified portions. These are all that are necessary, for the main object of the specifications is to obtain the weights of metal from which the principal diagrams of this paper were prepared. Should anyone desire to design nickel-steel bridges by the use of these specifications, he can do so by combining them properly with those of "De Pontibus." Moreover, if the result of this investigation is to bring nickel-steel bridges into favor, it is the writer's intention to prepare and make public complete specifications for designing such structures.

Metal.—All rolled steel shall be made by the basic open-hearth process.

In bridges composed entirely of nickel steel, the eye-bars, pins, and rollers shall be made of "eye-bar steel"; the rivets, bolts and all adjustable members shall be made of "rivet steel"; and all other portions, except castings, shall be made of "plate-and-shape steel."

* For a complete record of these tests see Appendix E of the original paper in the Society's Library. It contains Professor Hatt's report.

In bridges of mixed nickel and carbon steels, the floor system, the lateral system, and those parts of trusses of minor importance, such as struts having a large excess of section above the theoretical requirements, lacing bars, and stay-plates, may be made of carbon steel; but the floor system shall, preferably, be of nickel steel.

TABLE 4.—Composition of Rolled Steel.

Ingredients.	Percentages.		
	Rivet steel.	Plate-and-shape steel.	Eye-bar steel.
Nickel	3.50 (3.25 to 3.75)	3.50 (3.25 to 3.75)	4.25 (4.00 to 4.50)
Carbon	0.15 (0.12 to 0.18)	0.38 (0.34 to 0.42)	0.45 (0.40 to 0.50)
Phosphorus	0.03 maximum	0.03 maximum	0.03 maximum
Sulphur	0.04 maximum	0.04 maximum	0.04 maximum
Silicon	0.04 maximum	0.04 maximum	0.04 maximum
Manganese	0.60 (0.55 to 0.65)	0.70 (0.65 to 0.75)	0.80 (0.75 to 0.85)

As the manufacturer will have to keep the elastic limit and the ultimate strength up to certain minima, he will be allowed some liberty in the amounts of carbon to use in order to produce the required results, but he is not to attempt to obtain such results by increasing the manganese, nor will he, under any circumstances, be permitted to pass the limits of phosphorus, sulphur, or silicon; in fact, these limits should be kept as much below the specified amounts as practicable, because these elements are all detrimental to the metal. Preferably, the amounts of nickel should be kept within the limits set; but, in case of necessity, the latter, with the written permission of the Engineer, may be varied from.

Method of Determining Elastic Limit.—The elastic limit for specimen tests shall be assumed to be the load on the specimen producing a permanent set of 0.01 in. in a gauge length of 8 in., the amount of set being measured by fine dividers with no load on the specimen, or being taken from an autographic record at the intersection with the stress-strain curve of a line drawn parallel to and 0.01 in. away from the straight portion of the record.

The elastic limit for tests of full-sized eye-bars shall be assumed to be the load on the bar producing a permanent set of 0.04 in. in a gauge length of 20 ft.; or a proportionate set for shorter lengths, the amount of set being measured by an extensometer of approved design, with no load on the eye-bar.

Tensile Strength.—The ultimate tensile strength per square inch on unannealed test pieces for all three kinds of rolled nickel steel used in structural metalwork shall be as follows:

Rivet steel	70 000	to	80 000	lb.
Plate-and-shape steel	105 000	"	120 000	"
Eye-bar steel	115 000	"	130 000	"

The preceding figures are for test pieces taken from the edge of the piece. In case the test pieces are taken from the interior, these figures may be reduced by 3 000 lb. each.

The preceding figures also apply for all plates up to $\frac{7}{8}$ in. in thickness and for all shapes up to $\frac{5}{8}$ in. in thickness. For each additional $\frac{1}{8}$ in. in thickness the ultimate strength may be reduced 1 500 lb. per sq. in., down to an inferior limit of 95 000 lb., for the thickest eye-bar flats.

Elastic Limits.—The least allowable elastic limits per square inch obtained from unannealed test pieces shall be as follows:

Rivet steel	45 000	lb.
Plate-and-shape steel	60 000	"
Eye-bar steel	65 000	"

The preceding figures are for test pieces taken from the edge of the piece. In case the test pieces are taken from the interior, the figures may be reduced by 2 000 lb. each.

The preceding figures also apply for all plates up to $\frac{7}{8}$ in. in thickness and for all shapes up to $\frac{5}{8}$ in. in thickness. For each additional $\frac{1}{8}$ in. in thickness the elastic limit may be reduced 1 000 lb., down to a limit of 57 000 lb. per sq. in. for plate-and-shape and eye-bar steels.

Elongation.—The percentages of elongation shall be obtained from the unannealed test pieces after breaking on an original length of 8 in., in which length must occur the curve of reduction from stretch on both sides of the point of fracture. The least allowable elongations for the three kinds of rolled structural steel shall be as follows:

Rivet steel	25	per	cent.
Plate-and-shape steel	15	"	"
Eye-bar steel	12	"	"

The preceding percentages apply to plates, shapes, and flats $\frac{1}{2}$ in. thick or less. For thicker metal they are to be increased by unity for each increase of $\frac{1}{4}$ in. in thickness.

Bending Tests.—Specimens of rivet steel shall be capable of bending, by either pressure or hammering, to 180° and closing down flat upon themselves without cracking, when either hot or cold.

Specimens of plate-and-shape steel, when either hot or cold, shall be capable of bending by pressure 180° around a mandrel having a diameter equal to twice the thickness of the test piece, without showing signs of cracking on the convex side of the bend.

Specimens of eye-bar steel, when similarly treated, shall be capable of bending by pressure 90° around a mandrel having a diameter equal to three times the thickness of the test piece, without showing signs of cracking on the convex side of the bend.

Drifting Tests.—Punched rivet holes in plate-and-shape steel, pitched two diameters from a sheared edge, must stand drifting until their diameters are 40% greater than those of the original holes, and must show no signs of cracking the metal.

The total taper of the drift-pins used for the testing shall not exceed 1 in 12.

Fracture.—All broken test pieces for all three classes of steel, and all broken eye-bars must show a silky fracture of uniform color.

Full-Sized Eye-Bars.—Full-sized eye-bars must show an ultimate tensile strength per square inch for the various thicknesses of metal as follows:

1-in.	105 000 lb.
$1\frac{1}{2}$-in.	100 000 "
2-in.	95 000 "
$2\frac{1}{2}$-in. or greater..............	90 000 "

The elongation shall not be less than 10% in a gauged length of 10 ft.; and the elastic limit shall not be less than 55% of the ultimate strength of the bar.

Cast Steel.—All steel castings shall be made of open-hearth steel of the same composition as that specified for eye-bar steel, except that acid steel having a maximum limit for phosphorus of 0.06% may be used. The ultimate tensile strength shall vary within the limits of 110 000 and 130 000 lb. per sq. in.; the elastic limit shall not be less than 55% of the ultimate tensile strength; and the elongation of test specimens in 2 in. shall not be less than 20 per cent.

Impact Allowance Load.—The impact allowance load is to be a percentage of the equivalent uniform live load found by the following formulas:

$$P = \frac{40\,000}{L + 500} \text{ for railroad bridges,}$$

$$\text{and } P = \frac{10\,000}{L + 150} \text{ for highway bridges,}$$

where P is the percentage and L is the length, in feet, of span or portion of span covered by the live load, when the member considered is subjected to its maximum stress.

Intensities of Working Stresses.—The following intensities of working stresses (that is, pounds per square inch of cross-section) are to be used for all cases, except where wind loads are combined with other loads, in which cases the intensities are to be increased 25%; but the sections shall not be less than those required by the stresses from all loads except wind.

Tension on eye-bars	30 000 lb.
Tension on plates and shapes in bottom chords, main diagonals, and laterals	28 000 "
Tension on net section of plate-girder flanges (assuming one-eighth of the area of the web to act as part of each flange), on extreme fibers of rolled I-beams, and on shapes in body of suspenders, hip verticals, and hanger plates (there being 50% increase of net area for section through eyes)	24 000 "
Bending on pins	50 000 "
Bearing on pins (measured upon the projection of the semi-intrados on a diametral plane)	38 000 "
Bearing on rivets (measured similarly)	30 000 "
Shear on pins	25 000 "
Shear on rivets	14 000 "
Shear on webs of plate girders (gross section)	17 000 "

For field rivets, the intensities for bearing and shear are to be reduced 20 per cent.

Compression on top chords	$30\,000 - 120\frac{l}{r}$
Compression on inclined end posts	$30\,000 - 140\frac{l}{r}$
Compression on all other struts with fixed ends	$27\,000 - 120\frac{l}{r}$
Compression on all other struts with one or two hinged ends	$27\,000 - 160\frac{l}{r}$

where l is the unsupported length of the strut, in inches, and r is its least radius of gyration, in inches.

Compression on end stiffeners of plate girders...... 22 000 lb.

For forked ends, the intensity of working stress shall be determined by the formula,

$$p = 15\,000 - 450 \frac{l}{t}$$

where p is the greatest allowable intensity of working stress (impact being considered); l is the unsupported length, in inches, measuring from the center of the pin-hole to the center of the first transverse line of rivets beyond the point at which the full section of the member begins; and t is the total thickness of one jaw, in inches.

The greatest allowable pressure upon expansion rollers of fixed spans, when impact is considered, shall be determined by the equation,

$$p = 1\,000\ d,$$

where p is the permissible pressure, in pounds per linear inch of roller, and d is the diameter of the latter, in inches. The preceding formula is to be used for rollers of swing spans with the span at rest, but, for the span in motion, the formula to be used is

$$p = 400\ d,$$

where d is the mean diameter of the roller, in inches.

In order to anticipate criticism, the writer will now take each feature of the preceding specifications separately and show how the figures it contains were determined.

Composition of Rolled Steel.—The composition of the rivet steel was fixed by the numerous experiments made on various specimens of rivet nickel steel, most of which were objectionable in some particular. The essential requirements for a good rivet nickel steel are the following:

First.—That it shall flow readily, so as to fill the holes properly, and that it shall head without splitting;

Second.—That it is sufficiently soft to permit of the heads being cut off and the rivets being backed out without undue trouble and expense;

Third.—That it shall have sufficient strength as compared with rivet carbon steel to make its use instead of the latter a decided *desideratum.*

The rather wide limits of nickel, carbon, and manganese, of the specification, will assuredly permit manufacturers to fill these requirements.

The composition of the plate-and-shape steel is that specified for the two special melts manufactured for this investigation under the direct supervision of Mr. Colby; and, while it is not claimed that absolute perfection was attained by this first trial, it is not likely that any great improvement in the characteristics of the future plate-and-shape steel, as compared with those of these melts, will be effected. This steel is strong, tough, reliable, and capable of being manipulated in the shops without danger to the workmen, and without unduly great expense.

The satisfactory character of the composition of the eye-bar steel is not so well assured, as it was determined mainly by anticipation, the idea being to obtain as strong and hard a nickel steel as can be worked into eye-bars without running into brittleness.

The existence of a brittle zone in nickel steels has been lately claimed by Dr. H. C. H. Carpenter, Mr. R. A. Hadfield, and Mr. Percy Longmuir, who presented a paper on the subject to the Institution of Mechanical Engineers of England, in November, 1905.*

These gentlemen find that with from 40 to 50 points of carbon, and from 75 to 100 points of manganese, a steel containing 4½% of nickel has lost none of its toughness or ductility, as compared with steels having smaller percentages of nickel; but that a steel containing 5% of nickel has lost both to a serious extent. Between these two percentages there is probably a well-defined point of demarcation, but exactly where it is, can only be determined by further experiments. These should be made with the least possible delay.

They find also that with 7 or 8% of nickel the brittleness reaches a maximum, at 12% the resilience begins to increase, and at 20% the steel reaches its normal toughness. This is a very curious property of the alloy; and the knowledge acquired by these scientists, if corroborated by further experiments, will prove of great value to all metallurgists who are interested in the manufacture of nickel steel for bridges. The writer has heard, though, from good authority, that the best American practice in the manufacture of nickel steel does not agree with the findings of these experimenters, and that steel contain-

* A résumé of their investigations was published in *The Engineer* (London), November 24th, 1905.

ing 10% of nickel gave excellent results. On account of this disagreement of authorities, it is hoped that the discussion of this paper will bring out such a mass of evidence both *pro* and *con* that the existence or non-existence of the so-called brittle zone will be firmly established.

The recorded properties of the 4½% nickel steel experimented upon by these Englishmen were as follows:

Carbon	0.40 per cent.
Manganese	0.82 " "
Bending test (on an anvil)	180 degrees.
Yield point in tension	65 500 lb. per sq. in.
Ultimate strength in tension	109 100 " " " "
Elongation in 2 in	20 per cent.
Reduction of area	33 " "
Modulus of elasticity	29 900 000 lb.

The shock test, made by dropping a 46.7-lb. hammer about 14 in., showed that the 4½% nickel steel absorbed a greater amount of energy before breaking than did any of the other steels tested, the amounts of nickel therein varying from zero to 20 per cent. This shows that, as far as impact is concerned, there is no objection whatsoever to 4½% nickel steel for eye-bars.

Comparing the properties of this English 4½% nickel steel with those called for in the preceding specifications for eye-bar steel, it is seen that they strike an average for both nickel and manganese, touch the inferior limit for carbon, exceed somewhat the minimum elastic limit, and fall 6 000 lb. below the inferior limit for ultimate strength. Had the percentage of carbon been 0.45 instead of 0.40, this English steel would undoubtedly have had sufficient ultimate strength to meet the requirements of the specifications.

As for the fitness of 4½% nickel steel to be manufactured into eye-bars, this is proved by the fact that for this investigation two eye-bars were fabricated from ⅞-in. plates of 4½% nickel steel, as stated previously. These eye-bars contained 46 points of carbon and 67 points of manganese, as determined by the chemist of the Osborn Engineering Company; and the specimen tests gave elastic limits varying from 69 200 to 78 200 lb. (determined by the drop of the beam at medium testing speed), ultimate tensile strengths varying from 112 100 to

122 400 lb., percentages of elongation varying from 11.5 to 18.75 and averaging 16.8, and reductions of area varying (with one exception, where the fracture was unsatisfactory) from 39.5 to 47.1 per cent.

As previously stated, the two eye-bars made from this material gave ultimate strengths of 102 300 and 105 900 lb. per sq. in.; but, unfortunately, the elastic limit in both cases was missed. The elongation in 10 ft. was about 7%, and the reduction of area about 46 per cent.

It must be remembered that these were not *bona fide* eye-bars, for they were manufactured from universal mill plates with planed edges. Had the edges been rolled, instead of planed, it is almost certain that better results would have been found, especially in the elongation. The amount of manganese was less than the specifications call for. Had it been increased, so as to agree with them, the ultimate strength of the specimens would probably not have fallen below their requirements.

As it was, the metal of which these eye-bars were made complied with the specifications in respect to elastic limit, gave an average ultimate strength only 300 lb. per sq. in. below the lowest requirement, and failed only in one case out of fourteen to give the demanded elongation (and then only by one-half of 1 per cent.).

Considering that this was "picked-up" steel, and in view of the great superiority of Mr. Colby's low-nickel steel over the "picked-up" specimens of similar nickel steel, as shown by all the tests, it is fair to conclude that there will be no trouble whatsoever in obtaining eye-bar steel that will easily meet the preceding specifications, and at the same time will permit of satisfactory heads being forged with facility.

The matter of annealing nickel-steel eye-bars, in order to obtain the best possible results, is one worthy of a full and immediate investigation. The writer had intended to settle this question by experiments, but his failure to obtain a special melt of eye-bar steel prevented. It was his intention to test two thicknesses of eye-bars, namely, 2-in., and 2½-in., from 6 to 8 in. wide, and 6 ft. long, in groups of three, by annealing at slightly varying temperatures, in order to determine from the average elastic limits and ultimate strengths of each group what temperature will give the best result for each thickness of metal.

Should this series of tests on nickel steel for bridges ever be continued beyond the limits of this investigation, one of the first steps

would be to manufacture a special melt of eye-bar steel, make the usual specimen tests of the metal, so as to see that it is up to the requirements, prepare a full supply of short eye-bars of various thicknesses, and determine finally the best annealing temperature for each thickness.

The data concerning tests of nickel steel for the eye-bars of the Blackwell's Island Bridge, given in Appendix B, were obtained through the courtesy of the American Bridge Company. From them the writer endeavored to obtain, by averages, approximate rates of variation of elastic limit and ultimate strength with increased thickness of metal; but the attempt was a failure, as the results showed much irregularity, and really indicated that these characteristics of eye-bars are substantially independent of the thickness. Such a conclusion, however, would not be warranted, unless the metal of which all the eye-bars were composed was of practically uniform composition, which it was not, by any means, except in so far as the percentage of nickel was concerned.

In order to obtain, if possible, some general information of value from these Blackwell's Island Bridge tests, the writer has prepared averages of the chemical compositions; of the elastic limits, ultimate strengths, elongations, and reductions of area, of both unannealed and annealed specimens; and of the thicknesses, elastic limits, ultimate strengths, elongations, and reductions of area of full-sized bars, but, in making the averages, he rejected the elongations and reductions of area of those bars that broke in the eye. The results of these computations are as follows:

Chemical Analyses.

Carbon	0.39
Phosphorus	0.012
Sulphur	0.03
Manganese	0.71
Nickel	3.39

Unannealed Specimens.

Elastic limit	60 610 lb. per sq. in.
Ultimate strength	106 385 " " " "
Elongation	17.8% in 8 in.
Reduction of area	29.9%

Annealed Specimens.

Elastic limit	54 377 lb. per sq. in.
Ultimate strength	97 084 " " " "
Elongation	22.5% in 8 in.
Reduction of area................	42.9%

Full-Sized Eye-Bars.

Thickness	$1\frac{31}{32}$ in.
Elastic limit	50 006 lb. per sq. in.
Ultimate strength	88 818 " " " "
Elongation	12.2% in 18 ft.
Reduction of area..............	36.3%

Comparing these average results with the corresponding figures given in the preceding "Specifications for Nickel-Steel Bridges," the following conclusions are reached:

The impurities of phosphorus and sulphur are well within the limits of the specifications, the percentage of carbon is 6 points less, the percentage of manganese 9 points less, and the percentage of nickel 86 points less; but, if this eye-bar metal be compared with the plate-and-shape steel of the specifications, the percentages of carbon and manganese are found to be almost the same, and the percentage of nickel only 11 points less; consequently, the average metal of the Blackwell's Island Bridge eye-bars is almost identical in composition with the plate-and-shape steel of the specifications, and with the special melts made for the writer's experiments.

The average elastic limit and ultimate strength of unannealed specimens are very near the lower limits of the specifications for plate-and-shape steel, and considerably below those for eye-bar steel. Had the specifications for plate-and-shape steel been used for the manufacture of these eye-bars, and had advantage been taken of the clauses permitting the lowering of the elastic limit and ultimate strength with the increased thickness of metal, there would have been no special difficulty in complying with the specifications, as far as the testing of unannealed specimens is concerned.

Had the specifications for eye-bar metal been used for the manufacture of these eye-bars, there would have been numerous rejections because of elastic limit, but very few, if any, on account of ultimate strength, as far as the testing of unannealed specimens is concerned.

As for elongation of unannealed specimens, it is evident that there would have been very little difficulty in complying with the specifications for plate-and-shape steel, and none at all in complying with those for eye-bar steel.

In respect to the tests of full-sized eye-bars, the average ultimate strength fell some 6 000 lb. below the requirements of the specifications, and the average elastic limit about 2 500 lb. below. Of course, in individual instances, the discrepancies were much greater.

Had the eye-bar specifications been drawn on the assumption that plate-and-shape steel was to be used for eye-bars, the limiting ultimate strength for 2-in. bars would have been about 87 500 lb. per sq. in., and the corresponding elastic limit about 48 000 lb. per sq. in. In every test the elastic limit would have been complied with, but in 35% of the tests the ultimate strength would have fallen short, sometimes materially so, owing to the fact that many bars broke in the eye.

As for the elongation, no difficulty would have been found in complying with the specifications.

Comparing the differences in elastic limit and ultimate strength between unannealed and annealed specimens, the averages of all the tests show, respectively, 6 233 and 9 301 lb., the corresponding percentages being 10.3 and 8.7. The writer found in his experiments approximately 9 and 3, which is a fairly close agreement.

Concerning the differences in elastic limit and ultimate strength between unannealed specimens and full-sized eye-bars, the averages of all the tests show, respectively, 10 604 and 17 567 lb., the corresponding percentages being 17.5 and 16.4. The writer found in his experiments approximately 11 and 6. These discrepancies are mainly due to the fact that the inspector made his specimen test slowly, but possibly in part by more careful annealing of the eye-bars. No data were obtained from the American Bridge Company concerning the annealing of their nickel-steel eye-bars, but the writer hopes that some of its officers, in discussing this paper, will treat this point thoroughly.

Method of Determining Elastic Limit.—The method described in the specifications for determining the elastic limit will suffice while the building of nickel-steel bridges is still in its infancy, because at first there will be ample time for testing the new metal; but, afterward, when the manufacture of such structures from nickel steel becomes general, the testing will have to be done on a more commercial

basis, and the elastic limit will then have to be determined by the drop of the beam.

For such conditions, this specification is suggested:

"The elastic limit, in testing specimens, may be determined by the drop of the beam, according to the accepted practice in all steel mills, provided the speed of the machine or the movement of the head is not more than 1 in. in 3 min., up to the elastic limit, and that the weight is moved out by hand at a uniform speed sufficient to keep the beam in very light contact with the upper cross-piece. The speed, after the elastic limit is passed, shall not be greater than 1 in. in 30 sec. nor less than 1 in. in 1 min."

The true elastic limit may be determined as just described, provided the operator maintains a uniform movement of weight, as specified.

Little dependence can be placed on the elastic limits from specimen tests as usually made, because the speed of the machine is so rapid that the lever is kept pressed hard against the upper cross-bar, and thus the elastic limit is passed by 3 000 or 4 000 lb. per sq. in. before the beam has a chance to drop. The ultimate strength is exaggerated in the same manner, but not usually to the same extent as the elastic limit. In order to obtain proper and reliable records for all specimen tests, the limits of the speed, both before and after the elastic limit is passed, should be materially reduced, but not to such an extent as either to involve a hardship for the manufacturer or to cause the inspectors excessive labor or trouble.

Tensile Strength.—It has been shown previously, for this item, that there will be no special difficulty in filling the requirements of the specifications for the three kinds of nickel steel; especially if it be remembered that the figures given as limits may be reduced 3 000 lb. each for specimens taken from the interior, and that there is a sliding-scale reduction for specimens exceeding a certain thickness. With these limitations, manufacturers should have no serious difficulty in obtaining in all cases the ultimate strength called for.

Elastic Limits.—The preceding remarks upon the specification for ultimate strength apply to the elastic limit specification as well.

Elongation.—The specifications governing elongation are by no means difficult to fill; indeed, they are possibly not severe enough, and in the future it may be advisable to raise the percentages somewhat.

Bending Tests.—None of the requirements specified for bending are difficult to satisfy, provided the operator does not attack the metal brutally, but gives it a chance to show what it is worth by taking plenty of time and by seeing that the bend is not made too sharp.

Drifting Tests.—Nor are the drifting-test requirements difficult to satisfy, if the operator will not strike too hard blows and if he will turn the plate over quite often. The enlargement of the holes should always be made gradually and with care.

Fracture.—As practically all the test pieces in this series of tests complied with this requirement, there should be no difficulty in the future in living up to it.

Full-Sized Eye-Bars.—As it is not likely that nickel-steel eye-bars will be made less than 1½ in. or more than 2½ in. thick, it is not probable that there will be any special difficulty in obtaining ultimate strengths of 100 000 lb. for the smaller thickness, and 90 000 lb. for the greater thickness. The writer found, for the weakest of his 2-in. bars, an ultimate strength of 89 000 lb. per sq. in., and, as the metal specified for eye-bar steel is 10 000 lb. stronger in specimens than that of which his eye-bars were fabricated, there ought to be no difficulty in meeting the requirements.

The elastic limit in nickel steel never falls below 55% of the ultimate strength.

Cast Steel.—The specifications for cast steel are not difficult to fill, and if they should prove so in any particular it would be easy enough and perfectly legitimate to amend them, as castings form but a small portion of bridge material.

Impact Allowance Loads.—The impact allowance loads are the same as those given in "De Pontibus," and their reliability has never been questioned; for it is generally conceded that they are decidedly in excess of the real impact. It was the writer's intention, when the book was written, to make them cover, not only the actual impact, but also small, unavoidable secondary stresses and slight inequalities of stress distribution. The various intensities of working stresses adopted were properly adjusted to these impact allowances.

Tension on Eye-Bars.—There are several ways of checking the correctness of the 30 000 lb. per sq. in. for the working stress on eye-bars. One is to take the ratio of least allowed elastic limits for full-sized eye-bars of nickel steel and of carbon steel and multiply it by the

intensity given in "De Pontibus" for carbon-steel eye-bars, namely, 18 000 lb. The result would be:

$$\frac{49\,500 \times 18\,000}{28\,000} = 31\,800 \text{ lb.}$$

Another way is to use the ratio of least allowed ultimate strengths of full-sized eye-bars. The result would then be:

$$\frac{90\,000 \times 18\,000}{56\,000} = 28\,900 \text{ lb.}$$

Still another check is to use the ratio of least allowable elastic limits for specimen tests, which would give:

$$\frac{65\,000 \times 18\,000}{35\,000} = 33\,400 \text{ lb.}$$

Or, adopting the ratio of least allowable ultimate strengths of specimens, there results:

$$\frac{115\,000 \times 18\,000}{60\,000} = 34\,500 \text{ lb.}$$

Averaging these four results gives 32 100 lb., which is well above the 30 000 lb. specified.

Tension on Built Members.—The intensity given in the new specifications is 28 000 lb., and that for carbon steel from "De Pontibus" is 16 000 lb. Applying the ratio of least allowable elastic limits, gives:

$$\frac{60\,000 \times 16\,000}{35\,000} = 27\,400 \text{ lb.}$$

Or, taking the ratio of least ultimate strengths:

$$\frac{105\,000 \times 16\,000}{60\,000} = 28\,000 \text{ lb.}$$

It is true that the first check shows a deficiency of 600 lb., but this small amount is fully compensated for by the greater resistance of nickel steel to the abuse which all metal receives in the shops.

Tension on Net Section of Flanges of Beams, etc.—The intensity of tension on net section specified for nickel steel is 24 000 lb., and that for carbon steel is 14 000 lb. Applying the ratio of least elastic limits gives:

$$\frac{60\,000 \times 14\,000}{35\,000} = 24\,000 \text{ lb.}$$

Bending on Pins.—The ratio of bending resistances on pins for plate-and-shape nickel steel and carbon steel found by the tests was

1.85. As eye-bar steel is about 8% stronger, this ratio should be increased to 1.85 × 1.08 = 2.0. The working intensity for bending on pins of carbon steel is 27 000 lb., consequently, that for pins of high-nickel steel would be 2 × 27 000 = 54 000 lb., while the specifications call for only 50 000 lb. If the ratio of 1.85 were used, the intensity would be almost exactly 50 000 lb.

Bearing on Pins.—For bearing on pins, the writer found, for plate-and-shape steel compared with carbon steel, a ratio of 1.66. The intensity for carbon steel is 22 000 lb., consequently, the application of the ratio would give 22 000 × 1.66 = 36 500 lb. Owing to the superior stiffness of nickel steel, and because the intensity given in "De Pontibus" is rather low for bearing on pins as compared with the other intensities of the specifications, the intensity for bearing on nickel-steel pins was taken at 38 000 lb. It must be remembered that this is not calculated for the high steel of the pin, but for the lower steel of the bearing.

Bearing on Rivets.—The experiments show that for rivets the ratio of bearing stresses is 1.83. The intensity for carbon steel is 20 000 lb. Applying the ratio gives 20 000 × 1.83 = 36 600 lb. Another method of calculating this is to take the 38 000 lb. found for bearing on pins and multiply it by the ratio of least elastic limits of rivet nickel steel and plate-and-shape nickel steel, giving 38 000 × 45 000 ÷ 60 000 = 28 500 lb. As a compromise between these two widely varying results, a value of 30 000 lb. was adopted.

Shear on Pins.—As no experiments were made on shear on pins, the proper intensity can be taken by proportion from the established bending intensities for both nickel-steel and carbon-steel pins and from the given working shear of 15 000 lb. per sq. in. for carbon-steel pins, thus $S = 15\,000 \times 50\,000 \div 27\,000 = 27\,800$ lb. The amount adopted in the specifications is only 25 000 lb.

Shear on Rivets.—The writer found the comparing ratio for ultimate strengths of nickel-steel rivets and carbon-steel rivets in shear to be about 1.4. Applying this to the intensity of working stress for carbon-steel rivets, namely, 10 000 lb., gives 14 000 lb. for the intensity of shear on nickel-steel rivets, which is the figure adopted for the specifications.

Shear on Webs of Plate Girders.—The intensity of working stress for carbon steel is 10 000 lb.; and, as the ratio of elastic limits for

plate-and-shape steel and carbon steel is about 1.7, the intensity for shear on web plates of nickel steel should be 17 000 lb., as given in the specifications.

Compression Formulas.—As the column tests on nickel steel were limited to six, the data for preparing formulas were very meager, but the comparing ratios found by the writer for long and for short columns, namely, 1.47 and 1.75 (the ratios of length to least radius of gyration for the two cases being, respectively, 27 and 81), sufficed for the establishment of empirical formulas similar to those of "De Pontibus."

The short struts tested correspond to top-chord panel lengths, for which the carbon-steel formula is:

$$p = 18\,000 - 70\,\frac{l}{r}.$$

The formula assumed for nickel-steel top chords in the specifications is:

$$p = 30\,000 - 120\,\frac{l}{r}.$$

Testing this for $\frac{l}{r} = 30$, gives

$$p = 15\,900 \text{ for carbon steel,}$$
$$\text{and } p = 26\,400 \text{ for nickel steel.}$$

The ratio of these values is 1.65, instead of 1.75 as shown by the experiments.

For $\frac{l}{r} = 50$, which is the usual limit for top-chord sections of railroad bridges,

$$p = 14\,500 \text{ for carbon steel,}$$
$$\text{and } p = 24\,000 \text{ for nickel steel.}$$

The ratio of these values is 1.65. By interpolation from the experiments, this would have been about 1.64.

These examples show that the new top-chord formula errs on the side of safety for small values of $\frac{l}{r}$, and is just right for the usual values in top chords.

The formula for inclined end posts of nickel steel was obtained approximately from that found for the top chords by simply varying the coefficient of $\frac{l}{r}$ by the ratio of the corresponding coefficients in the formulas for carbon-steel struts.

For all other nickel-steel columns with fixed ends the formula assumed in the specifications was:

$$p = 27\,000 - 120\,\frac{l}{r},$$

while that for carbon-steel struts is:

$$p = 16\,000 - 60\,\frac{l}{r}.$$

The usual average value of $\frac{l}{r}$ for such struts is about 80, for which the formulas give:

$$p = 17\,400 \text{ for nickel steel,}$$
$$\text{and } p = 11\,200 \text{ for carbon steel.}$$

The ratio of these values is 1.55, which is somewhat more than the ratio given by the experiments for hinged ends; but it would probably be amply safe for fixed ends.

The formula for all other nickel-steel struts with one or two hinged ends was derived from the formula for similar struts with fixed ends by modifying the coefficient of $\frac{l}{r}$ in about the same ratio as exists in the two carbon-steel column formulas, namely, $80 \div 60 = 1.33$. Thus, $120 \times 1.33 = 160$; which is the constant adopted in the nickel-steel column formula for hinged ends.

Testing this for $\frac{l}{r} = 80$ gives:

$$p = 14\,200 \text{ for nickel steel,}$$
$$\text{and } p = 9\,600 \text{ for carbon steel.}$$

The ratio of these values is 1.48, which agrees almost exactly with the results of the experiments.

As soon as nickel steel is actually used for building bridges, it will become necessary to make some additional experiments on columns with both fixed and hinged ends for various lengths so as either to establish some new formulas or to verify the preceding ones. Meanwhile, it will be perfectly safe to use the latter in designing nickel-steel bridges.

Compression on End Stiffeners of Plate Girders.—The intensity for carbon-steel stiffeners is 14 000 lb., and as $\frac{l}{r}$ for end stiffeners is small, say not to exceed 40, the safe ratio to use will be 1.68, hence the intensity for nickel-steel end stiffeners could be $14\,000 \times 1.68 = 23\,500$ lb., while the specifications call for only 22 000 lb.

Formula for Forked Ends.—As $\frac{l}{r}$ in forked ends is likely to run as high as 80, the safe ratio will be about 1.5. Applying this to the carbon-steel formula, namely,

$$p = 10\,000 - 300\,\frac{l}{t},$$

$$\text{gives } p = 15\,000 - 450\,\frac{l}{t},$$

which is the formula adopted for forked ends of nickel steel.

Formula for Expansion Rollers.—The carbon-steel formula for expansion rollers is $p = 600\,d$. Applying the general ratio of strength, namely, 1.7, gives for the constant $1.7 \times 600 = 1\,040$; hence the formula for nickel-steel rollers at rest was made $p = 1\,000\,d$.

For rollers in motion, this was changed to $p = 400\,d$.

The only specification previously written for nickel steel in bridges, so far as the writer knows, is that of R. S. Buck, M. Am. Soc. C. E., for the Manhattan Bridge, issued by the Department of Bridges of the City of New York. In it the following intensities are specified for nickel-steel members for a combination of dead load, temperature, and congested live load, or for a combination of dead load, regular live load, temperature, and wind.

	Pounds per square inch.
Tension in stiffening trusses	40 000
Compression in stiffening trusses	$40\,000 - 150\,\frac{l}{r}$
Shear on rivets in stiffening trusses (field)	20 000
Bearing on rivets in " " "	35 000

As the writer's proposed specifications are for a combination of dead load, live load, impact-allowance load, and wind load, his allowance for impact, for the purpose of comparison, may be allowed to offset Mr. Buck's allowance for temperature. For this combination, the writer's intensities of working stresses would be as follows:

	Pounds per square inch.
Tension in stiffening trusses (eye-bars)	37 500
Tension in stiffening trusses (shapes)	35 000
Compression in stiffening trusses (chords)	$37\,500 - 150\,\frac{l}{r}$
Compression in stiffening trusses (webs), fixed ends	$33\,750 - 150\,\frac{l}{r}$

Compression in stiffening trusses (webs), hinged ends	$33\,750 - 200\,\frac{l}{r}$
Shear on rivets in stiffening trusses (field)	14 000
Bearing on rivets in stiffening trusses (field)	30 000

Comparing the two sets of figures, it will be seen that Mr. Buck has in every case stressed his nickel steel higher than the writer has stressed his, notwithstanding the fact that the requirements for strength of metal in Mr. Buck's specifications are decidedly less than in those of the writer. It is evident, therefore, that, as compared with the *dicta* of the sole present authority on the use of nickel steel in bridgework, the specifications herein presented for the designing of nickel-steel bridges err assuredly on the side of safety. It is true that Mr. Buck's specifications are for a very long span, and in consequence his intensities of working stresses are permitted to run high by the best established engineering practice; but it must be remembered that the writer's intensities, by the impact allowance of his specifications, are adjusted properly for spans of all lengths.

In making this comparison the writer is not endeavoring to criticise Mr. Buck's specifications, but is simply anticipating possible criticism of his own on the plea of overstraining the nickel steel.

On Figs. 10 to 21, inclusive, are given the weights of all ordinary single-track and double-track railway bridges for all spans up to 1 800 ft. There are included four types of cantilever bridges, namely, *A, B, C,* and *D,* as shown on Fig. 22. Type *A* is the most usual, and is suitable where only one long span is necessary. Type *B* is for a bridge of very great length, where two long main spans are required. Type *C* is for the case where the total length is greater than for Type *A*, but short as compared with that for Type *B*. As regards economy of metal, Type *C* comes next to Type *A*. Type *D* is intermediate between Types *B* and *C*. These four types cover all the possible layouts of spans for cantilever bridges, or at least all that are consistent with good engineering practice.

Class *R* of the "De Pontibus" specifications was adopted as the live load for simple-span bridges; and, for cantilever structures, Class *R* was used for the stringers, Class *S* for the floor-beams and the primary truss members, and Class *U* for the main truss members. All

the cantilever structures were assumed to have double tracks, as such bridges are now rarely, if ever, built with single tracks.

For the purpose of record, all lay-outs for cantilever bridges were assumed to have the following constant proportions between the lengths of their various spans:

Calling l the length of the main span of Type A, $\frac{3}{8} l$ will be the length of the suspended span and $\frac{5}{16} l$ that of each cantilever arm and of each anchor arm. For the anchor span, when there is one, the length will be $\frac{5}{8} l$. These proportions are all shown correctly to scale on Fig. 22. The plotted weights of metal per linear foot of span for the cantilever bridges are the average weights for the entire length of structure.

The weights of metal per linear foot of span for the carbon-steel bridges were computed by using the specifications of "De Pontibus," and those for the nickel-steel and mixed-steel bridges by the specifications of this paper combined with those of "De Pontibus." These weights are as accurate as they can well be made, and much time was spent by the writer's office force in calculating them. At some future time, after bridge building in nickel steel has been inaugurated, the writer will give to the Profession curves of weights of metal per linear foot in nickel-steel, mixed-steel, and carbon-steel bridges for all his standard live loads; but, for the present, those offered in this paper will have to suffice.

From the weights of metal per linear foot, given on Figs. 10 to 21, inclusive, from various assumed pound prices of carbon-steel bridges erected, and from various assumed differences in pound prices of superstructure metal delivered at site, in nickel steel and in carbon steel, the costs in dollars per linear foot of span (for bridges of all types in "all nickel steel," "mixed steel," and "carbon steel") were plotted, and are given in Figs. 23 to 72, inclusive.*

The range of pound prices for carbon steel erected is from 2.5 to 5.5 cents for plate-girder bridges, and from 3.0 or 3.5 cents to 6.0 cents for all other bridges. These ranges are likely to include, for many years, all the conditions of the market for carbon-steel bridges erected;

* Only 50 of 154 diagrams are reproduced in this paper. The others may be seen in the Society's Library. These 50, however, are all that will usually be needed in the immediate future when comparing the costs of carbon-steel bridges with those of bridges built of nickel steel in all parts where the adoption of the alloy is economical, and of carbon steel in all other parts.

although a combination of general prosperity and a distant and difficult location might cause the superior limits to be passed occasionally. Such a contingency, however, is too remote to warrant the preparation of more diagrams than those that accompany this paper, especially since, in such a case, it would require only a few minutes to make by proportion the necessary correction for the special proposed structure.

The differences in pound prices of nickel steelwork and carbon steelwork delivered at site range from 0.6 cent to 2.0 cents. These ought to be sufficient, for when the difference becomes as low as 0.6 cent, these tables will have served their purpose and become obsolete, because then practically all bridges will be built of nickel steel; and even to-day a variation of 2 cents would indicate an excessive overcharge on the part of the manufacturers. In the case of the Manhattan Bridge, the difference bid by the contractors between nickel-steel eye-bars and carbon-steel eye-bars erected was 1.5 cents per lb.; consequently, the difference for the steels delivered at site would have been somewhat less than that amount. It is true that the difference in cost of manufacture of entire bridges in nickel steel and in carbon steel is somewhat greater than the corresponding difference in the case of eye-bars alone, but the variation is certainly not so great as ½ cent per lb. If a greater difference than 2 cents per lb. between the values of nickel steelwork and carbon steelwork delivered at site should occur in any case, it would be an easy matter to plot a curve to meet the condition by proportionate extension on the special diagram that is to be used.

The plotted pound prices for carbon-steel bridges erected vary by ½ cent per lb. In case it is desired to assume any price intermediate between those on the diagrams, the comparison between costs per foot of nickel-steel, mixed-steel, and carbon-steel bridges erected should be made, first, for the next greater price and then for the next lower one, after which the necessary interpolation would be a simple matter.

The proportion of the total cost of the erected metal which pertains to the erection has been arbitrarily assumed for convenience at 20 per cent. This gives a range of from 0.5 cent to 1.2 cents per lb. as the cost of erection, which is a fair assumption, for, even in elevated-railroad work, the cost of erection and painting is seldom as low as ½ cent per lb., and for railroad bridges, even in remote localities, it does not

often exceed 1.2 cents per lb. For cantilever bridges, the cost of erection might be more, and thus a comparison of cost slightly too favorable to nickel steel might be made, were it not for the fact that in such large structures the use of this metal will have a tendency to lower comparatively the pound cost of erection, because the decrease in weight of individual members facilitates progress and reduces the cost of the traveler, derricks, and other heavy apparatus. On the whole, the assumption made for the proportionate cost of erection is, perhaps, the fairest that could be adopted.

In comparing the costs of erection of nickel-steel bridges and carbon-steel bridges due cognizance was taken of the fact that, for two similar bridges of equal carrying capacity, while the total cost of erection of the nickel-steel structure is less than that for the carbon-steel structure, the cost per pound in the former is greater than in the latter, because certain items of expense are constant while others vary with the weight of metal handled. The writer has assumed that one-half the total expense is constant and that the other half will vary directly with the weight of metal. This is as accurate a division as can be assumed. Upon this basis was established the following mathematical statement:

Let W = weight of metal per linear foot of span in the carbon-steel bridge,

W' = ditto for the nickel-steel bridge,

C = cost per pound for erecting the carbon-steel bridge,

C' = ditto for the nickel-steel bridge,

F = cost per linear foot for erecting the nickel-steel bridge,

then CW = cost per linear foot for erecting the carbon-steel bridge,

$$F = \frac{C\,W}{2}\left(1 + \frac{W'}{W}\right),$$

$$C' = \frac{F}{W'} = \frac{CW}{2W'}\left(1 + \frac{W'}{W}\right) = \frac{C}{2}\left(\frac{W}{W'} + 1\right).$$

In plotting the curves of cost of nickel-steel bridges and mixed-steel bridges erected, given on the diagrams, the cost per pound of the erection of the alloy was computed by this last equation.

The types of bridges covered by the diagrams are as follows:

Single-track, deck, plate-girder spans,
Single-track, half-through, plate-girder spans,
Single-track, through, riveted, Pratt-truss spans,
Single-track, through, pin-connected, Pratt-truss spans,
Single-track, through, pin-connected, Petit-truss spans,
Double-track, through, riveted, Pratt-truss spans,
Double-track, through, pin-connected, Pratt-truss spans,
Double-track, through, pin-connected, Petit-truss spans,
Double-track, through, pin-connected, "Type *A*," cantilever bridges,
Double-track, through, pin-connected, "Type *B*," cantilever bridges,
Double-track, through, pin-connected, "Type *C*," cantilever bridges,
Double-track, through, pin-connected, "Type *D*," cantilever bridges.

These twelve types (barring double-track, plate-girder spans) cover practically all the bridges that are built nowadays in the United States. In the case of any type not included in the preceding list, the diagrams may be used by adopting that one for the structure most like it and for the existing conditions of the metal market. The differences between Types *A, B, C,* and *D* of cantilever bridges, illustrated on Fig. 22, were explained previously. In the diagrams of weights of these cantilever bridges it must not be forgotten that the weights of metal per linear foot of bridge given are the averages from end to end of structure, and are not the weights per foot for any particular span or spans.

In computing the weights of metal for various spans which are plotted on Figs. 10 to 21, inclusive, it was found, as might readily have been anticipated, that the economic truss and girder depths are somewhat less for nickel-steel than for carbon-steel bridges. The reasons for this are: first, that, in comparison with carbon steel, nickel steel can be strained higher in the compression members of chords than in those of webs; and, second, that in the webs of nickel-steel

bridges there are necessarily more minimum sections used than in those of carbon-steel bridges.

It is evident that, on account of both the smaller economic depths and the higher intensities of working stresses, the deflections of nickel-steel spans will be greater than those of corresponding carbon-steel spans. However, this increase of deflection is not a matter of any great importance.

If, eventually, nickel steel should supplant carbon steel in bridge-work, the latter metal will continue to be used for a long time in parts that do not take direct stress (such as stay-plates, lacing bars, and web stiffeners), in the lateral systems of all bridges, except those of extremely long spans; and in truss members having much larger sections than the stresses call for, such as web members near mid-span and the secondary vertical posts of Petit trusses.

In some cases, especially when bridge metal is cheap, a still further saving might be effected by making the entire floor system of carbon steel; but as the amount of money thus gained in the floor would be small, and as it would have to be reduced somewhat by the increased cost of trusses due to the slightly greater dead load, this kind of economy is problematical. In long-span bridges the necessity of keeping the dead load as low as possible would preclude the adoption of carbon steel for the floor system, even if the use of such steel there were *per se* decidedly the cheaper.

In all the diagrams it has been assumed that carbon steel would be used exclusively for lateral systems; but it is a fact that in long spans it would be economical to adopt nickel steel for some of the heavier lateral members, consequently, the comparative costs of long-span bridges, of carbon-steel and of mixed-steel, in case of actual designs with careful detailing, might show even greater differences than those given by the curves.

In order to demonstrate how the diagrams are to be used, it will be well to assume a few cases and apply the curves to their solution.

Case 1.—A long, single-track bridge consists of a succession of half-through, plate-girder spans of 100 ft. each, carbon steel erected costing 4.5 cents per lb., and nickel steel delivered at site being worth 1.6 cents per lb. more than carbon steel. Find the comparative costs of carbon-steel and mixed-steel bridges.

Turning to Fig. 31, there are found \$101.50 as the cost per linear foot of the metal in the carbon-steel bridge, and \$92 as the corresponding cost for the mixed-steel bridge.

Case 2.—A double-track bridge consists of four riveted, through spans of 200 ft. each, carbon steel erected costing 4 cents per lb., and nickel steel delivered at site being worth 1.4 cents per lb. more than carbon steel. Find the comparative costs of carbon-steel and mixed-steel bridges.

Turning to Fig. 51, there are found \$188 as the cost per linear foot of the metal in the carbon-steel bridge, and \$174 as the corresponding cost for the mixed-steel bridge.

Case 3.—A double-track, Type *A*, cantilever bridge has a main span of 1 050 ft. If built of carbon steel, it would cost 5.5 cents per lb. erected. Nickel steel delivered at site is worth 1.5 cents per lb. more than carbon steel. Find the comparative costs of carbon-steel and mixed-steel bridges.

Turning to Fig. 69, there are found the following:

Carbon-steel bridge	\$658
Mixed-steel bridge for excess of 1.6c.............	555
" " " for excess of 1.4c.............	543
By interpolation for excess of 1.5c...............	549

An attempt will be made to anticipate what will be the probable excess pound price for shopwork on nickel steel as compared with carbon steel, using as a basis the present average cost of reamed shopwork, which is approximately $\frac{8}{10}$ cent per lb. in the principal American bridge shops. This figure includes all expenses and fixed charges of every kind, such as heat, light, power, and office expenses.

To obtain a result that is closely accurate, it will be necessary to itemize the various shop costs for carbon-steel work, add to each item its *pro rata* share of general expense, determine for each item the approximate ratio of increase for nickel-steel work, and calculate the increased items and increased total cost per pound.

The division of cost given in Table 5 is probably as good an average as could be assumed; but it must be remembered that any division whatsoever would vary with the style and individuality of the shops,

with the character of the construction, and even with the personnel of the shop management. Table 5 gives all the information required for obtaining the cost of reamed shopwork on nickel steel for bridges.

Each of the ratios of increased cost given in Table 5 was carefully considered in consultation with John Lyle Harrington, M. Am. Soc. C. E., and the items of division of shop cost were furnished by him.

Table 5 shows that the excess cost per pound for the manufacture of nickel-steel bridges, as compared with carbon-steel bridges, is 0.15 cent. Curiously enough, this is exactly the figure named to the writer four years ago as an off-hand guess by C. C. Schneider, Past-President, Am. Soc. C. E.

TABLE 5.—Comparison of Cost of Shopwork per Pound for Carbon Steel and Nickel Steel.

Items.	Cost of shopwork per pound for carbon steel.	Ratio of increased cost for nickel steelwork.	Cost of shopwork per pound for nickel steel.
Drawing-room work	0.08 cent.	1.25	0.100 cent.
Template-shop work	0.04 "	1.25	0.050 "
Laying-out work	0.04 "	1.10	0.044 "
Shearing and straightening	0.04 "	1.10	0.044 "
Punching	0.08 "	1.25	0.100 "
Assembling and bolting	0.12 "	1.10	0.132 "
Reaming and drilling	0.15 "	1.10	0.165 "
Chipping and milling	0.02 "	1.50	0.030 "
Riveting	0.16 "	1.30	0.208 "
Painting	0.03 "	1.25	0.037 "
Miscellaneous	0.04 "	1.00	0.040 "
Total and average	0.80 cent.	1.19	0.950 cent.

Nickel-steel ingots, when nickel is worth 30 cents per lb., cost about 1 cent per lb. more than those of carbon-steel; and the difference in cost of rolling should certainly not exceed $\frac{1}{10}$ cent per lb. Allowing 20% profit on these excess costs would make the total excess cost of nickel-steel bridgework delivered at site 1.5 cents per lb. The almost exact agreement of this difference with that for the eye-bars in the Manhattan Bridge appears to be pretty conclusive.

Adopting, then, 1.5 cents as the probable difference in pound prices of nickel steelwork and carbon steelwork delivered at site, it will be

interesting to compare the costs of bridges of carbon-steel and of mixed-steel of all the twelve kinds covered by the diagrams.

In making this comparison, it will be assumed that the average pound prices for carbon-steel bridges erected throughout the United States are as follows:

Plate-girder spans	4.0	cents.
Riveted-truss spans	4.5	"
Pin-connected, Pratt-truss spans	4.5	"
Pin-connected, Petit-truss spans	5.0	"
Cantilever bridges	5.5	"

The reason for the greater assumed pound costs of long-span bridges is mainly expensive erection, because such spans are generally used where the erection conditions are costly.

TABLE 6.—PERCENTAGES OF EXCESS OF COST OF CARBON-STEEL BRIDGES OVER MIXED-STEEL BRIDGES.

Type of Structure.	Least.	Greatest.	Approximate average.
Single-track, deck, plate-girder spans	— 5	+ 11	+ 5
Single-track, half-through, plate-girder spans	+ 3	+ 12	+ 7
Single-track, through, riveted, Pratt-truss spans	+ 2	+ 7	+ 5
Single-track, through, pin-connected, Pratt-truss spans	+ 1	+ 11	+ 6
Single-track, through, pin-connected, Petit-truss spans	+ 10	+ 17	+ 14
Double-track, through, riveted, Pratt-truss spans	+ 4	+ 8	+ 6
Double-track, through, pin-connected, Pratt-truss spans	+ 2	+ 11	+ 6
Double-track, through, pin-connected, Petit-truss spans	+ 13	+ 20	+ 16
Cantilever bridges of "Type *A*"	+ 7	+ 30	+ 18
Cantilever bridges of "Type *B*"	+ 9	+ 29	+ 19
Cantilever bridges of "Type *C*"	+ 7	+ 25	+ 18
Cantilever bridges of "Type *D*"	+ 12	+ 26	+ 20
General average for all bridges	+ 5	+ 17	+ 12

From Fig. 25 it is found that for deck, plate-girder spans, carbon-steel bridges are cheaper than mixed-steel bridges only for spans of less than 33 ft., and that in all greater spans they are more expensive.

A study of Figs. 30, 36, 41, 47, 52, 57, 63, and 69 shows that, for the conditions assumed, carbon-steel bridges are invariably more expensive than those of mixed nickel and carbon steels. The percentages of the greater cost are given in Table 6.

Summarizing, it is evident that it would be economical at the present time to use nickel steel for all kinds of railroad bridges, and the longer the spans the greater the economy. It might be shown also that nickel steel would be economical for certain highway bridges, but its adoption would certainly be inadvisable for ordinary county bridges, because the use of the new metal might cause such structures to "vanish into thin air."

The general use of nickel steel for bridges not only would result in decidedly cheaper structures, but also would permit of the building of longer spans than are at present attainable.

For instance, it is generally conceded by bridge engineers that the present greatest practicable main-span length for cantilevers built of carbon steel is in the neighborhood of 2 000 ft. On Fig. 71 are plotted the probable weights of metal per linear foot of bridge in carbon steel and in nickel steel for main spans far longer than any yet designed or computed. The method adopted for plotting was to record the already diagrammed weights for spans of 1 200 ft., 1 350 ft., 1 500 ft., 1 650 ft., and 1 800 ft., pass through the five points of each record a circular curve, and carry that curve to the limits of the paper. The weights of metal thus established for spans of unprecedented length are probably fairly accurate. In any case they are sufficiently so for the present purpose, which is simply to determine approximately the main span lengths of cantilever bridges of "Type *A*" that have the same average weight of metal per linear foot for the two kinds of steel. The diagram gives the following corresponding main-span lengths:

Carbon-steel bridges.........	1 200 ft.	1 400 ft.	1 600 ft.	1 800 ft.	2 000 ft.
Nickel-steel bridges..........	1 650 "	1 830 "	2 040 "	2 300 "	2 600 "

From this it will be seen that if 1 800 ft. be assumed as the present practicable limit of span length for carbon-steel bridges, the corresponding limit for nickel-steel bridges will be about 2 300 ft.; or, if it be assumed at 2 000 ft., the corresponding limit for nickel-steel construction will be 2 600 ft. It is safe, therefore, to conclude that the adoption of nickel steel for bridges would lengthen the practicable

span length for cantilevers fully 500 ft. The writer foretold this result just before the experiments on nickel steel for bridges were inaugurated.

If the question of greatest span length be one of economics instead of practicability, the curves on Fig. 72 should be used. These were prepared from the data on Fig. 69, assuming that the excess value of nickel steel delivered at site is 1.5 cents per lb.

The diagram gives the following corresponding lengths of main spans for equal costs per linear foot of bridge:

Carbon-steel bridges.........	1 200 ft.	1 400 ft.	1 600 ft.	1 800 ft.	2 000 ft.
Nickel-steel bridges..........	1 406 "	1 600 "	1 800 "	2 010 "	2 265 "

The principal application of the results of this last investigation is to the comparative economy of cantilever and suspension bridges; for if it be known that for carbon-steel construction the length of cantilever main span corresponding to equal cost per foot be 2 000 ft., it can be seen from the table that for nickel-steel construction it is 2 265 ft.

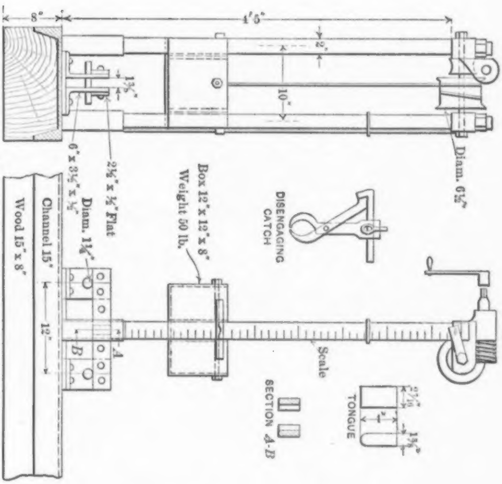
Summarizing the results of this entire investigation of nickel steel for bridges, it is evident that nickel steel is in every way fitted for bridge construction, in that it is strong, tough, workable, and reliable; moreover, its adoption would effect a decided economy. This economy would increase in the future as the cost of nickel decreases and as the shops become more accustomed to the fabrication of the new alloy. That nickel will soon be less expensive is a foregone conclusion, in view of the immense deposits of nickel ore that have been located and surveyed in Canada. It is said upon good authority that there has been found in one deposit in that country ore containing fully 200 000 tons of the metal.

While the writer has never known nickel to have been sold for less than 30 cents per lb., nevertheless, he is of the opinion that, should this material be called for in large tonnages for bridge building, it might be purchased as low as 25 cents. It makes a great difference in the price to the producer whether a metal is sold by the pound or by the ton; and tons of nickel would be required where pounds are bought today, were nickel steel used extensively for bridgework.

At 25 cents per lb. for nickel, the price of rolled nickel steel would be about 0.2 cent per lb. lower than it would be with nickel at 30 cents

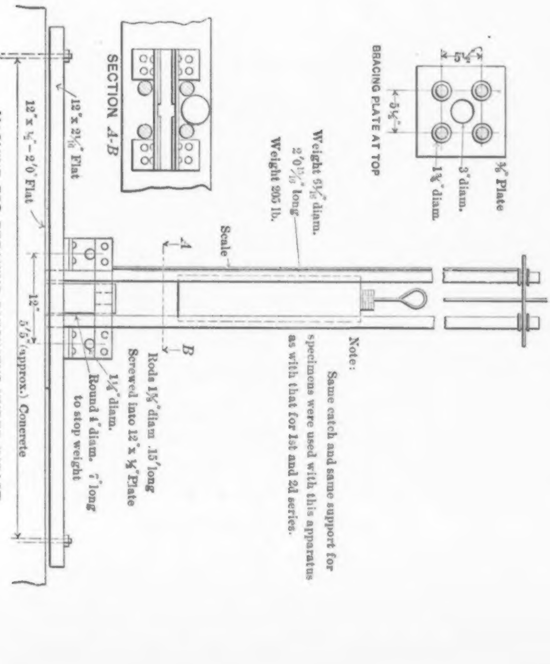
per lb., and it is likely that the cost of manufacture would be reduced in the same proportion, thus making the price of nickel-steel bridge metal delivered at site 1.2 cents per lb. above that of carbon steelwork. A study of the accompanying diagrams will show the great economy of using nickel steel for bridges under such conditions.

In concluding this paper the writer desires to ask for a thorough discussion, and to express the hope that the effect of the paper and the discussion will be to hasten materially the adoption of nickel steel for bridges.



MACHINE FOR BREAKING SPECIMENS UNDER IMPACT
1ST AND 2D SERIES

FIG. 1.



MACHINE FOR BREAKING SPECIMENS UNDER IMPACT
3D SERIES

FIG. 2.

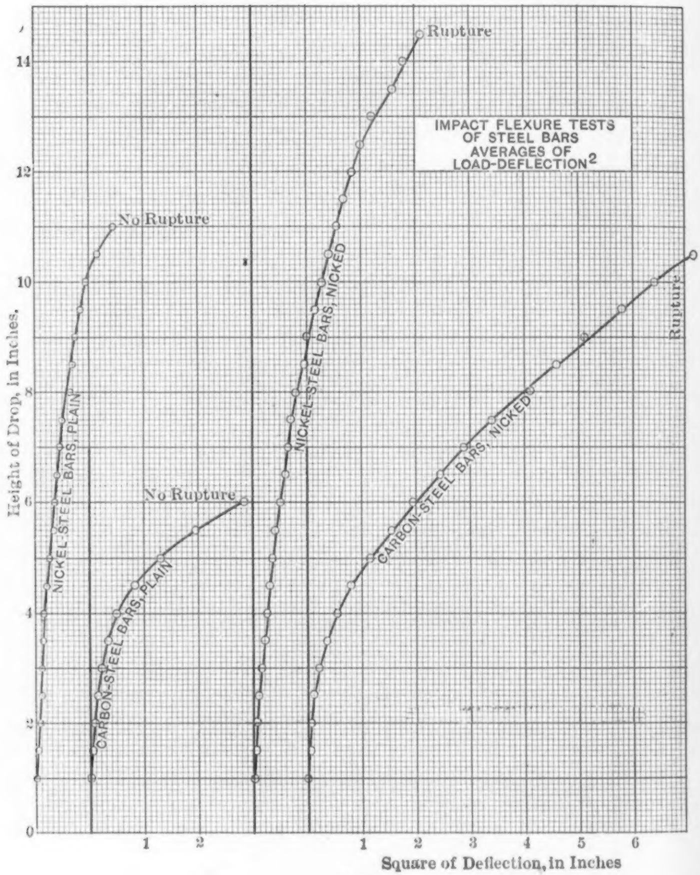

IMPACT FLEXURE TESTS
OF STEEL BARS
AVERAGES OF
LOAD-DEFLECTION²
Rupture
No Rupture
No Rupture
Rupture
NICKEL-STEEL BARS, PLAIN
CARBON-STEEL BARS, PLAIN
NICKEL-STEEL BARS, NICKED
CARBON-STEEL BARS, NICKED
Height of Drop, in Inches.
Square of Deflection, in Inches
0
2
4
6
8
10
12
14
1
2
1
2
3
4
5
6

Fig. 3.

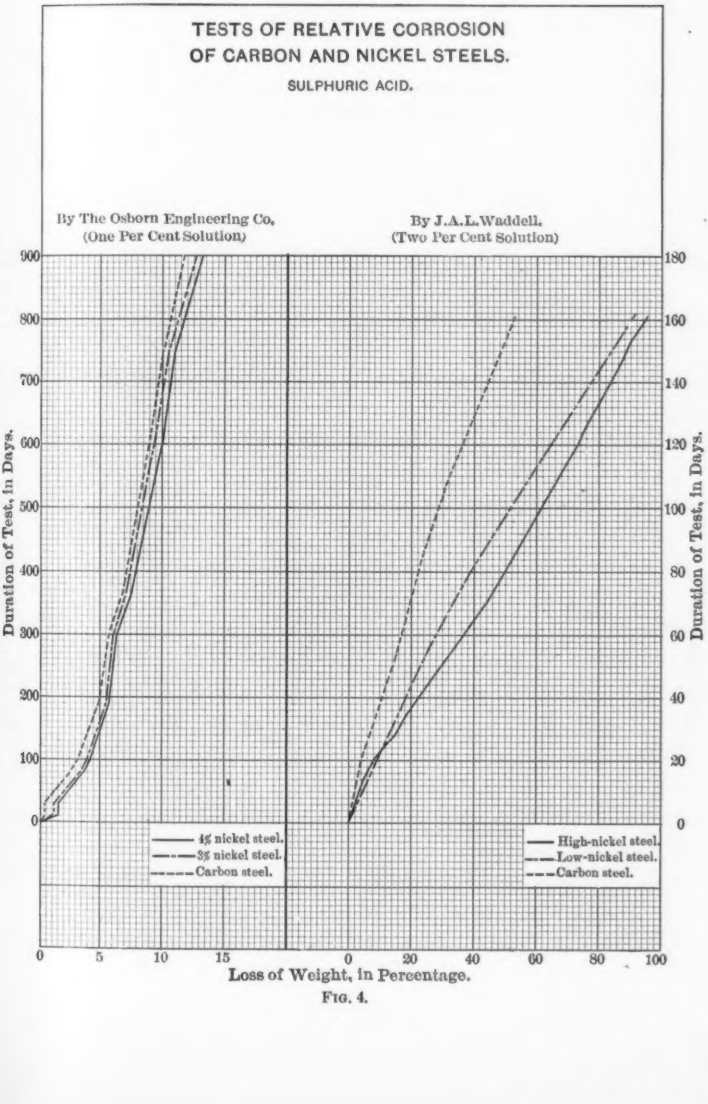

TESTS OF RELATIVE CORROSION
OF CARBON AND NICKEL STEELS.
SULPHURIC ACID.
By The Osborn Engineering Co.
(One Per Cent Solution)
By J.A.L.Waddell.
(Two Per Cent Solution)
Duration of Test, in Days.
0
100
200
300
400
500
600
700
800
900
4% nickel steel.
3% nickel steel.
Carbon steel.
High-nickel steel.
Low-nickel steel.
Carbon steel.
Duration of Test, in Days.
0
20
40
60
80
100
120
140
160
180
0
5
10
15
0
20
40
60
80
100
Loss of Weight, in Percentage.

FIG. 4.

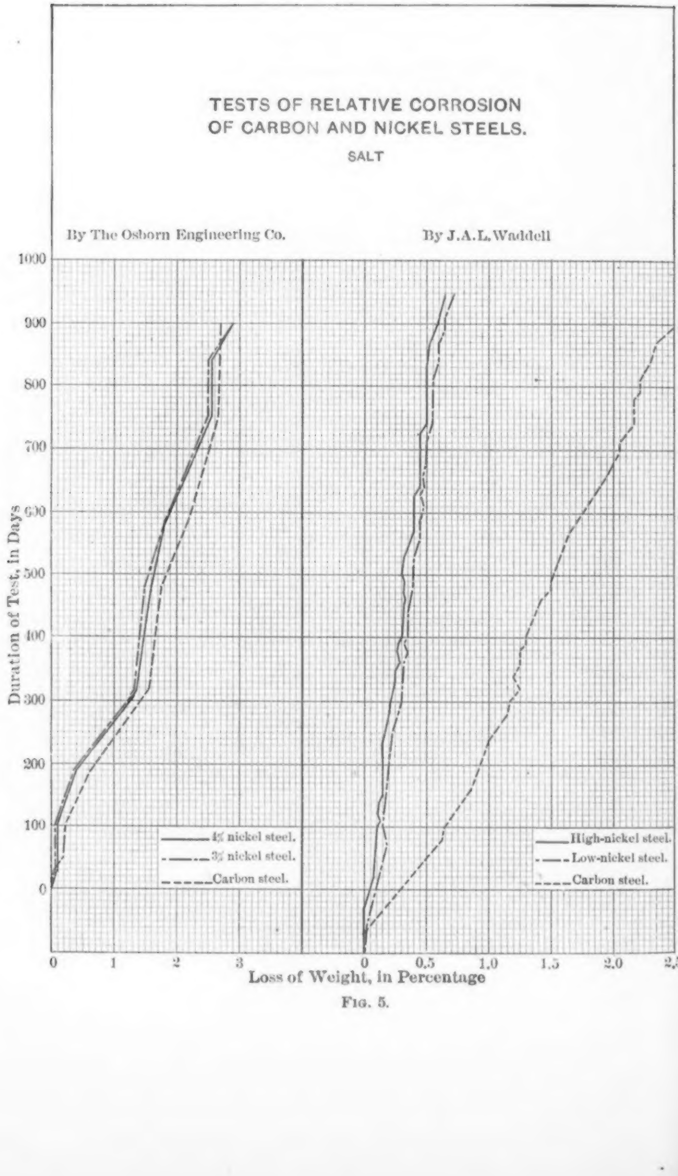

TESTS OF RELATIVE CORROSION
OF CARBON AND NICKEL STEELS.
SALT
By The Osborn Engineering Co.
By J.A.L. Waddell
Duration of Test, in Days
Loss of Weight, in Percentage
4% nickel steel.
3% nickel steel.
Carbon steel.
High-nickel steel.
Low-nickel steel.
Carbon steel.

FIG. 5.

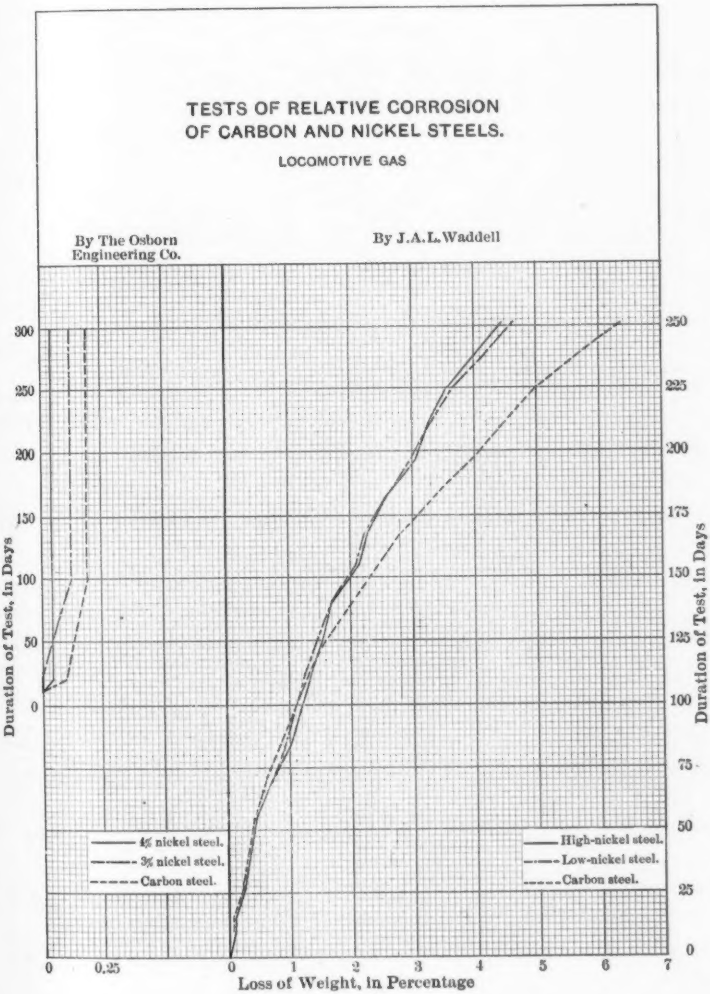

TESTS OF RELATIVE CORROSION
OF CARBON AND NICKEL STEELS.
LOCOMOTIVE GAS
By The Osborn
Engineering Co.
By J.A.L. Waddell
Duration of Test, in Days
300
250
200
150
100
50
0
Duration of Test, in Days
250
225
200
175
150
125
100
75
50
25
0
4% nickel steel.
3% nickel steel.
Carbon steel.
High-nickel steel.
Low-nickel steel.
Carbon steel.
0
0.25
0
1
2
3
4
5
6
7
Loss of Weight, in Percentage

Fig. 6

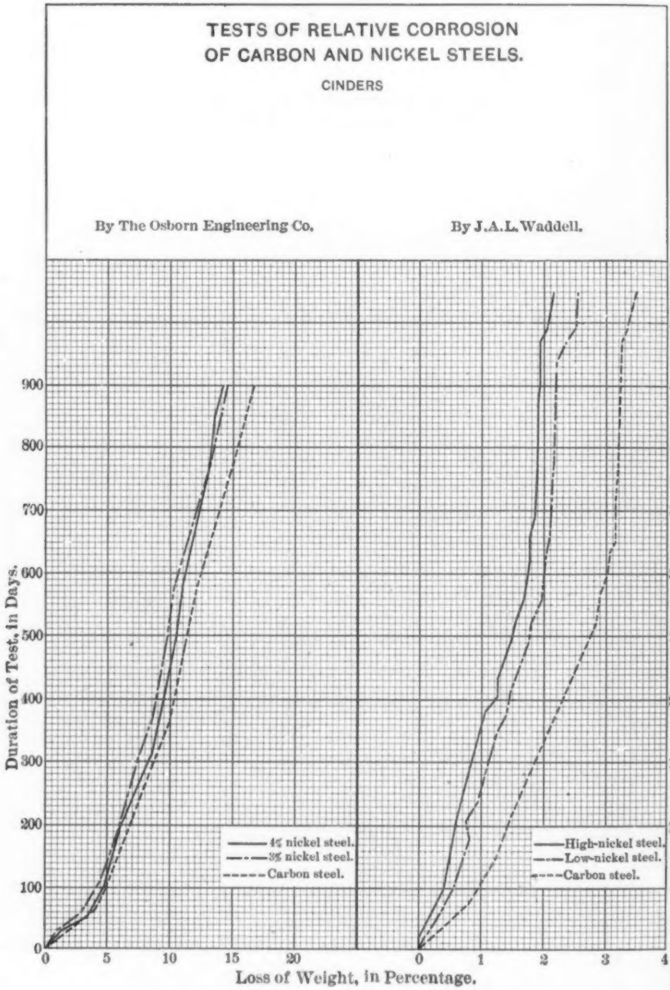

TESTS OF RELATIVE CORROSION
OF CARBON AND NICKEL STEELS.
CINDERS
By The Osborn Engineering Co.
By J.A.L.Waddell.
Duration of Test, in Days.
Duration of Test, in Days.
0
100
200
300
400
500
600
700
800
900
0
50
100
150
200
250
300
350
400
450
500
550
4% nickel steel.
3% nickel steel.
Carbon steel.
High-nickel steel.
Low-nickel steel.
Carbon steel.
0
5
10
15
20
0
1
2
3
4
Loss of Weight, in Percentage.

FIG. 7.

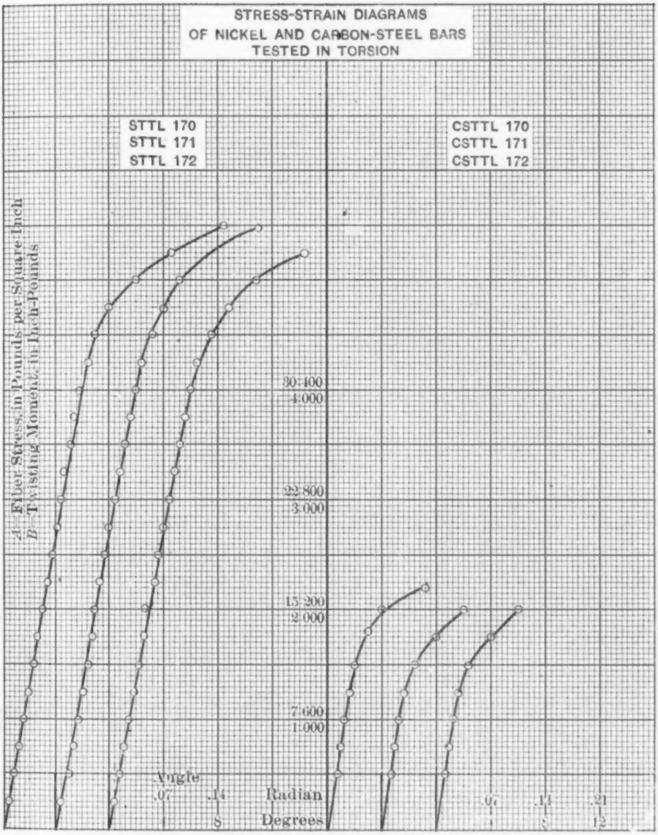

STRESS-STRAIN DIAGRAMS
OF NICKEL AND CARBON-STEEL BARS
TESTED IN TORSION
STTL 170
STTL 171
STTL 172
CSTTL 170
CSTTL 171
CSTTL 172
A—Fiber Stress, in Pounds per Square Inch
B—Twisting Moment, in Inch-Pounds
38 000
5 000
30 400
4 000
22 800
3 000
15 200
2 000
A-7 600
B-1 000
0
Angle
.07
.14
Radian
8
Degrees
.07
.14
.21
4
8
12

FIG. 8.

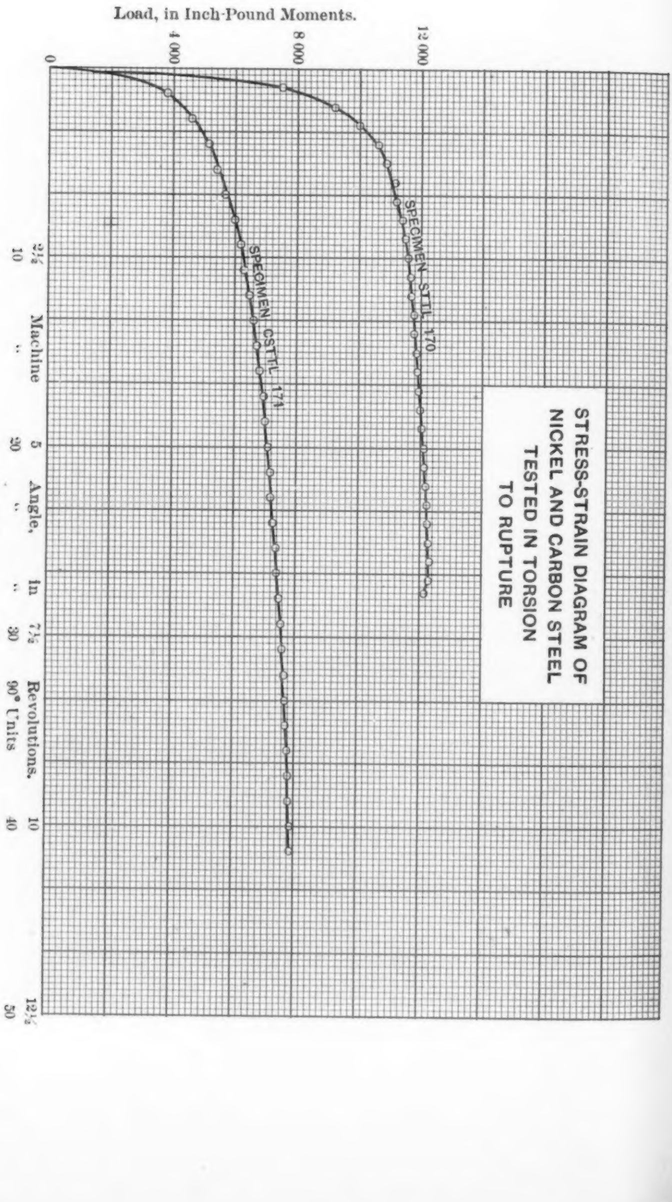

Load, in Inch-Pound Moments.
0
4 000
8 000
12 000
STRESS-STRAIN DIAGRAM OF NICKEL AND CARBON STEEL TESTED IN TORSION TO RUPTURE
SPECIMEN STTL 170
SPECIMEN CSTTL 171
Machine " Angle, " in Revolutions.
90° Units
2½
5
7½
10
12½
10
20
30
40
50

Fig. 9.

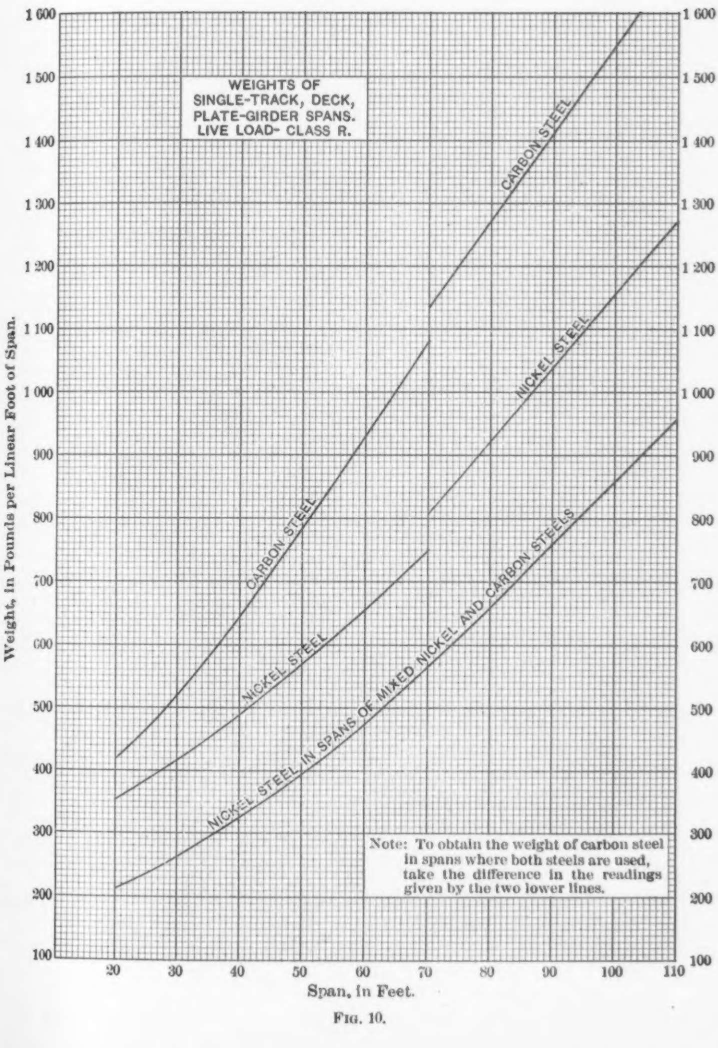

WEIGHTS OF
SINGLE-TRACK, DECK,
PLATE-GIRDER SPANS.
LIVE LOAD- CLASS R.
CARBON STEEL
NICKEL STEEL
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Span.
Span, in Feet.
1 600
1 500
1 400
1 300
1 200
1 100
1 000
900
800
700
600
500
400
300
200
100
20
30
40
50
60
70
80
90
100
110

FIG. 10.

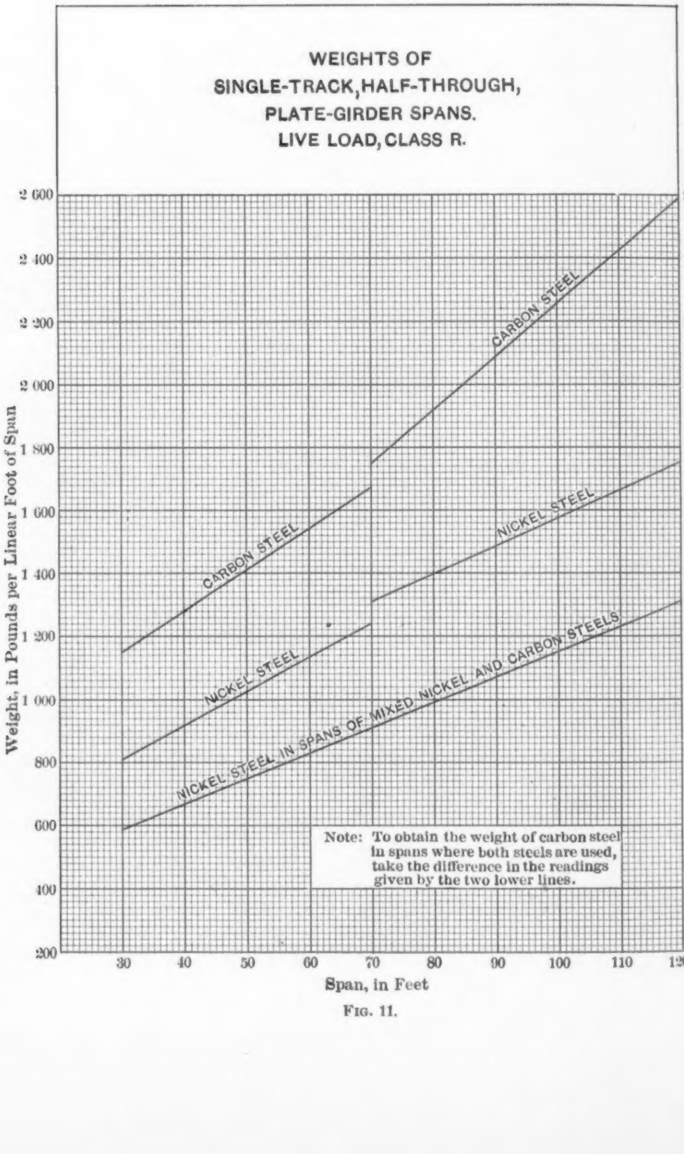

WEIGHTS OF
SINGLE-TRACK, HALF-THROUGH,
PLATE-GIRDER SPANS.
LIVE LOAD, CLASS R.
CARBON STEEL
NICKEL STEEL
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Span
2 600
2 400
2 200
2 000
1 800
1 600
1 400
1 200
1 000
800
600
400
200
30
40
50
60
70
80
90
100
110
120
Span, in Feet

FIG. 11.

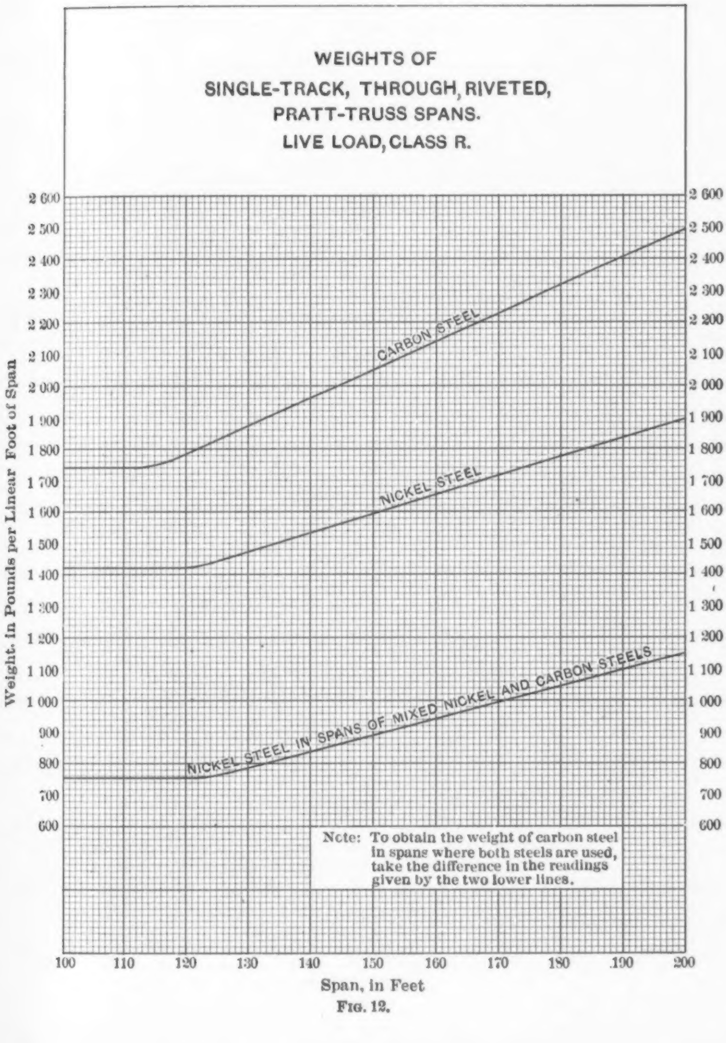

WEIGHTS OF
SINGLE-TRACK, THROUGH, RIVETED,
PRATT-TRUSS SPANS.
LIVE LOAD, CLASS R.
Weight. in Pounds per Linear Foot of Span
2 600
2 500
2 400
2 300
2 200
2 100
2 000
1 900
1 800
1 700
1 600
1 500
1 400
1 300
1 200
1 100
1 000
900
800
700
600
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
100
110
120
130
140
150
160
170
180
190
200
Span, in Feet

Fig. 12.

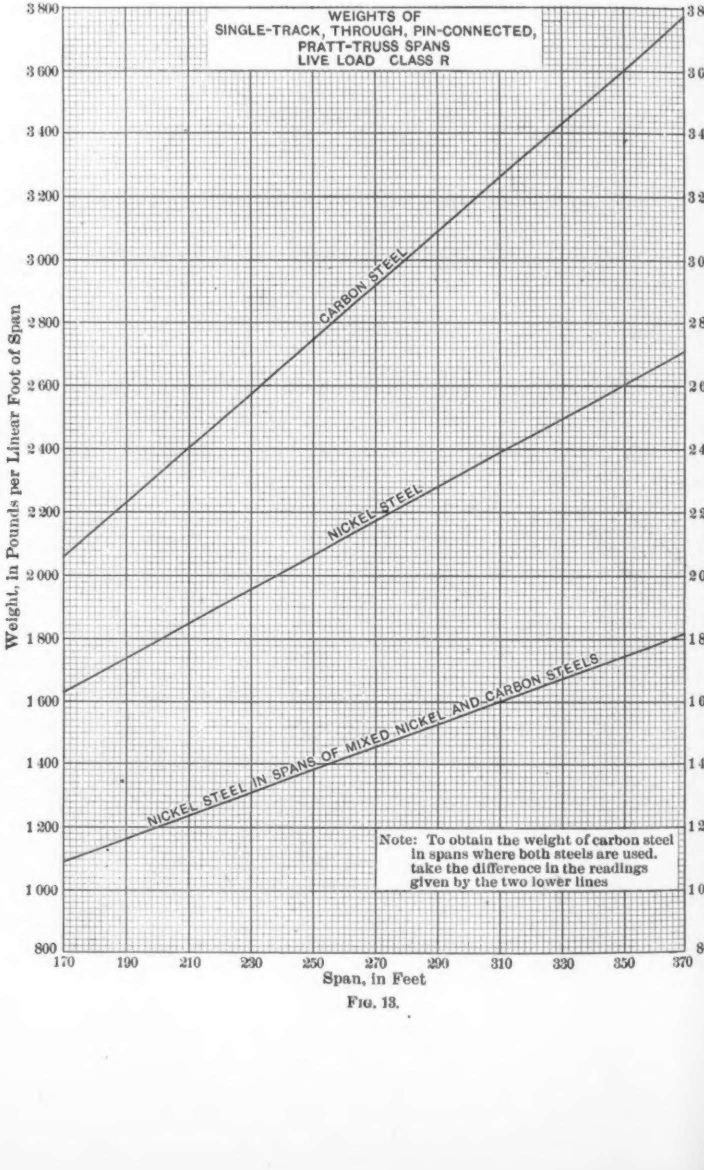
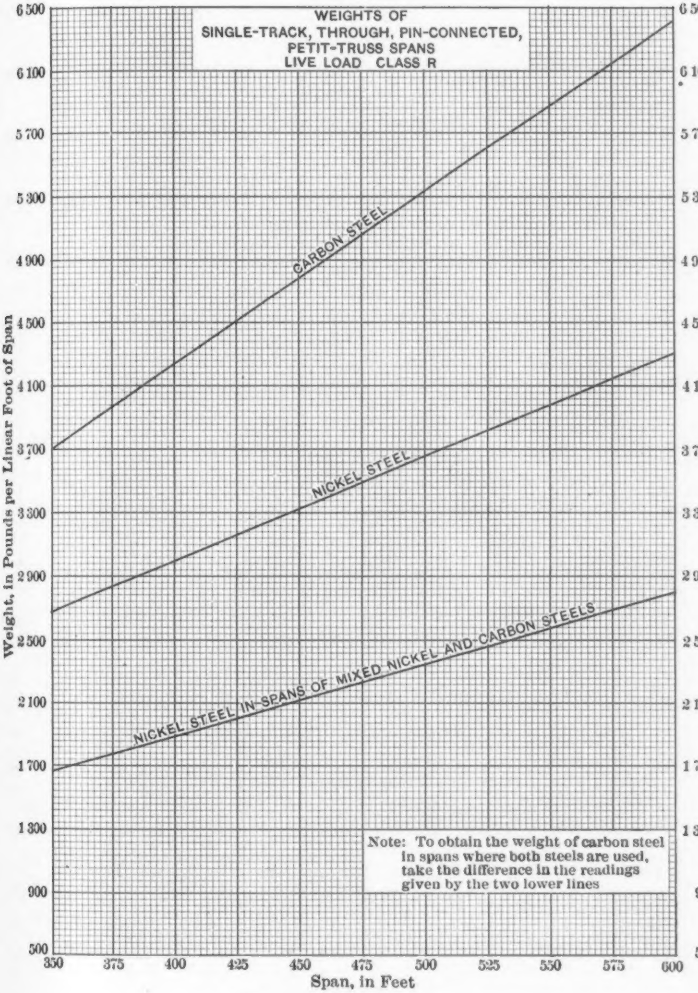
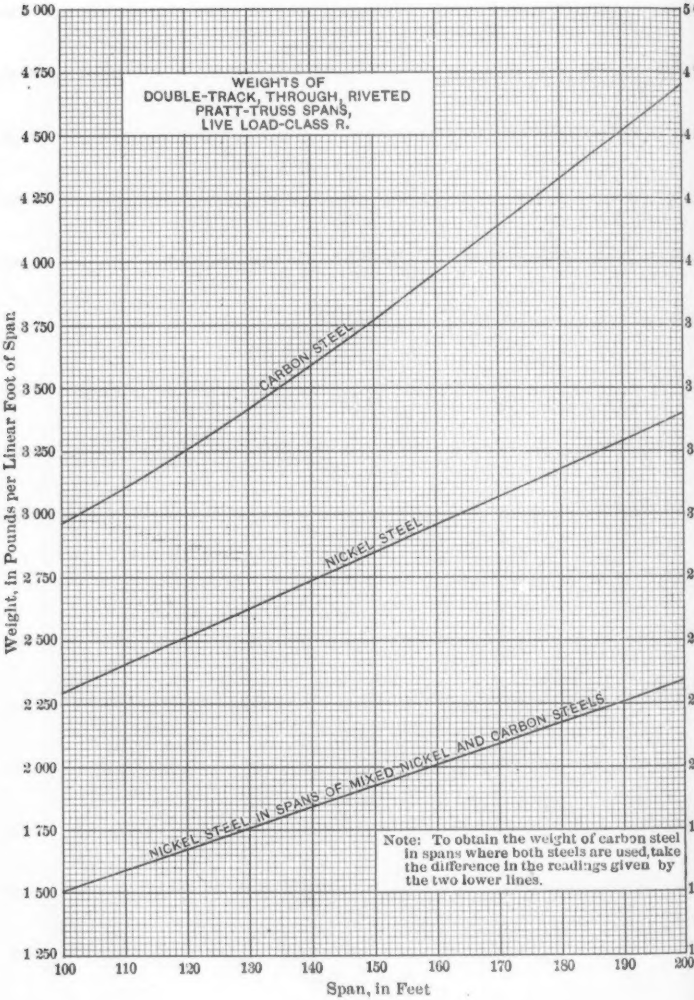

WEIGHTS OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS
LIVE LOAD CLASS R
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines
Weight, in Pounds per Linear Foot of Span
Span, in Feet
3 800
3 600
3 400
3 200
3 000
2 800
2 600
2 400
2 200
2 000
1 800
1 600
1 400
1 200
1 000
800
170
190
210
230
250
270
290
310
330
350
370

FIG. 13.


WEIGHTS OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS
LIVE LOAD CLASS R
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines
Weight, in Pounds per Linear Foot of Span
6 500
6 100
5 700
5 300
4 900
4 500
4 100
3 700
3 300
2 900
2 500
2 100
1 700
1 300
900
500
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet
FIG. 14.



WEIGHTS OF
DOUBLE-TRACK, THROUGH, RIVETED
PRATT-TRUSS SPANS,
LIVE LOAD-CLASS R.
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Span
5 000
4 750
4 500
4 250
4 000
3 750
3 500
3 250
3 000
2 750
2 500
2 250
2 000
1 750
1 500
1 250
100
110
120
130
140
150
160
170
180
190
200
Span, in Feet

FIG. 15.

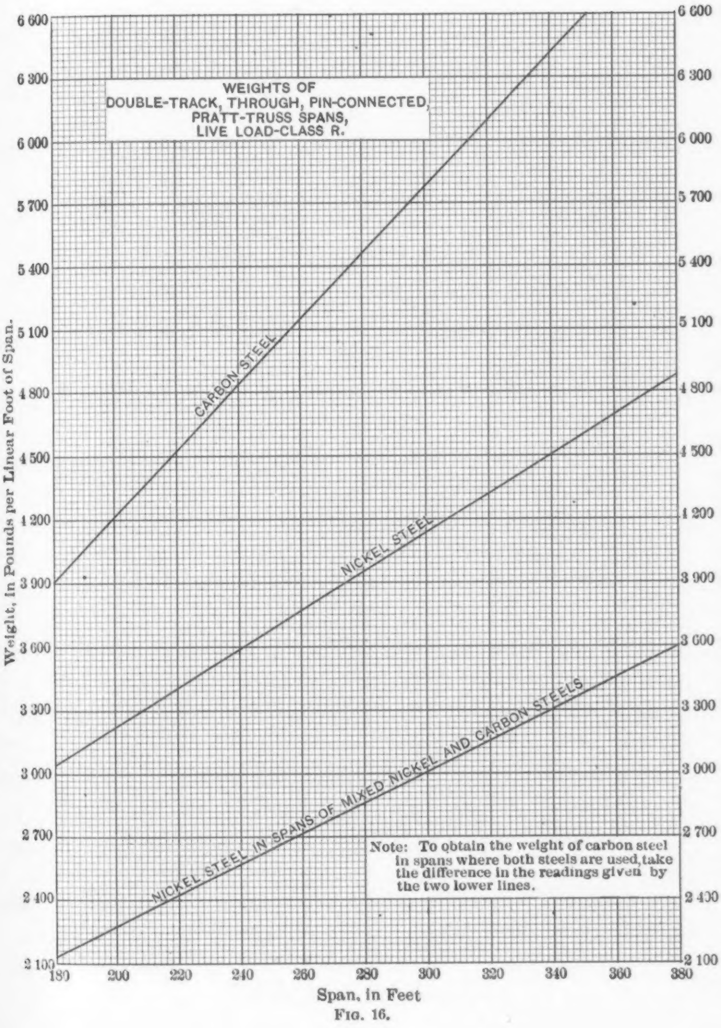

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS,
LIVE LOAD-CLASS R.
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Span.
Span, in Feet
6 600
6 300
6 000
5 700
5 400
5 100
4 800
4 500
4 200
3 900
3 600
3 300
3 000
2 700
2 400
2 100
180
200
220
240
260
280
300
320
340
360
380

Fig. 16.

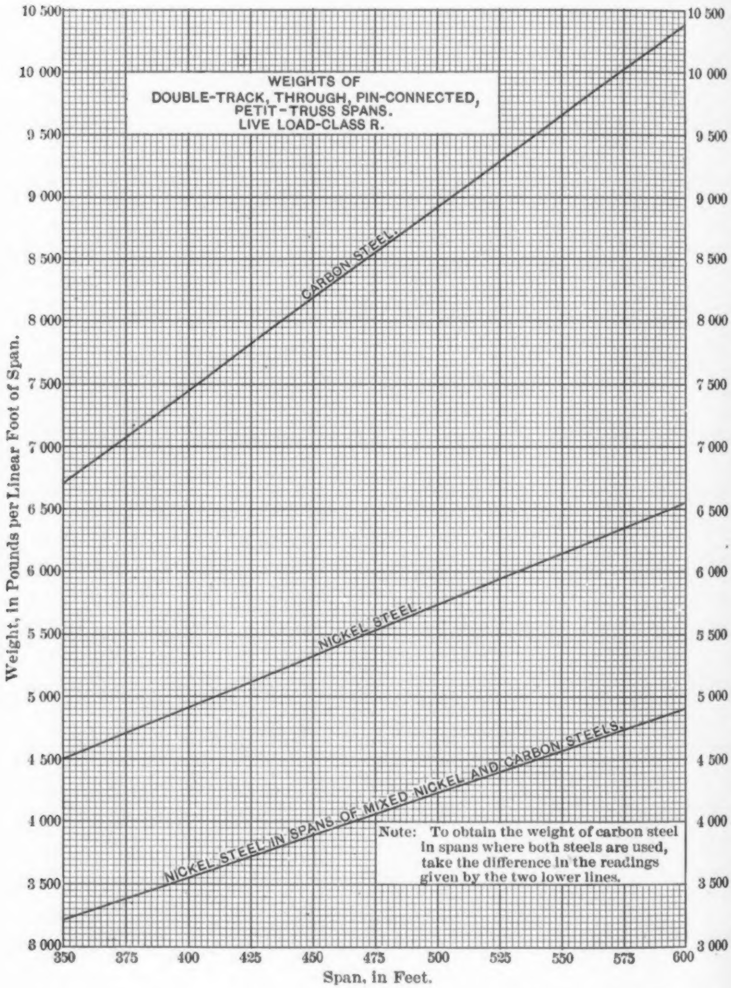

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS.
LIVE LOAD-CLASS R.
CARBON STEEL.
NICKEL STEEL.
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS.
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Span.
10 500
10 000
9 500
9 000
8 500
8 000
7 500
7 000
6 500
6 000
5 500
5 000
4 500
4 000
3 500
3 000
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet.

FIG. 17.

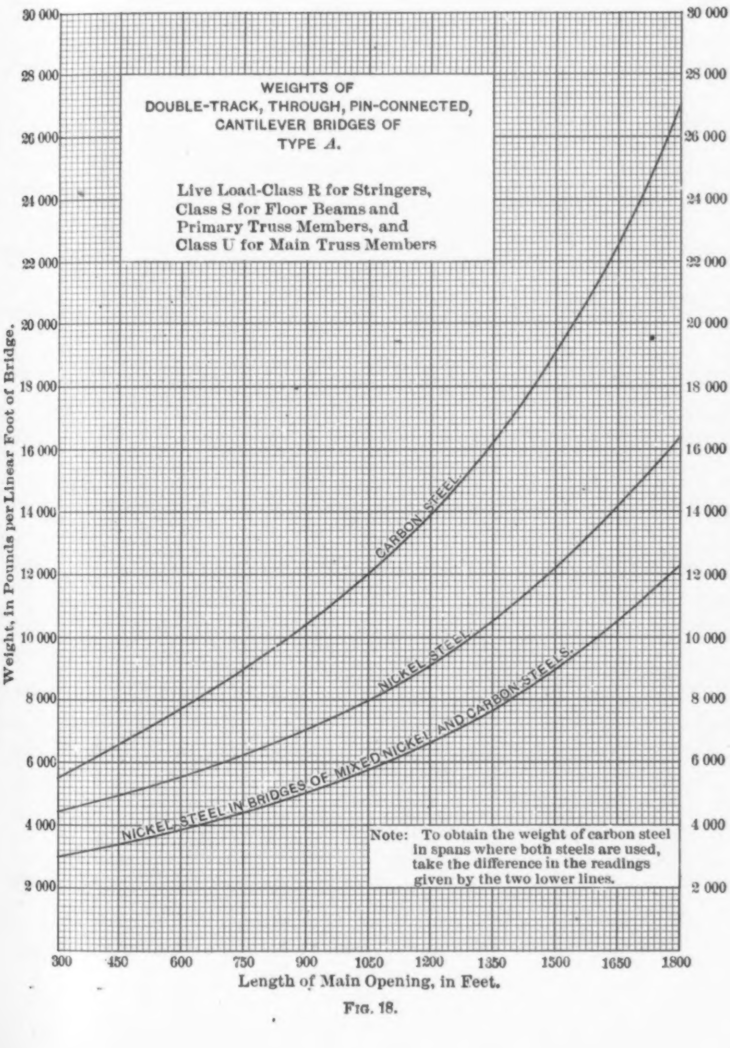

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
CANTILEVER BRIDGES OF
TYPE *A*.
Live Load-Class R for Stringers,
Class S for Floor Beams and
Primary Truss Members, and
Class U for Main Truss Members
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN BRIDGES OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Bridge.
30 000
28 000
26 000
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8 000
6 000
4 000
2 000
300
450
600
750
900
1050
1200
1350
1500
1650
1800
Length of Main Opening, in Feet.

FIG. 18.

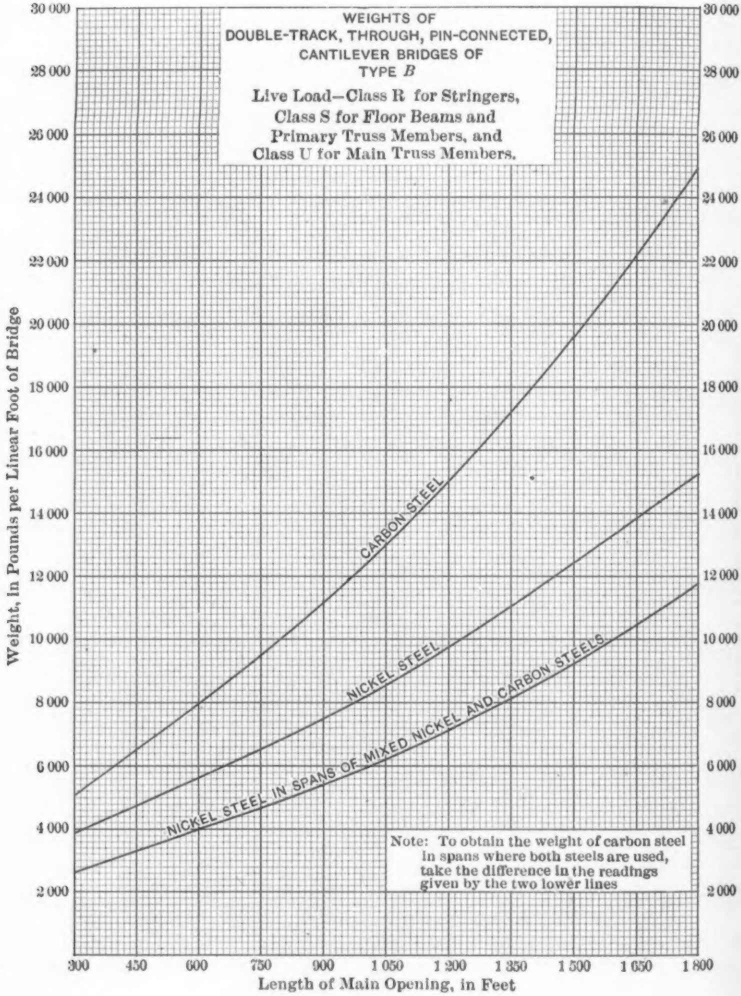

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
CANTILEVER BRIDGES OF
TYPE B
Live Load—Class R for Stringers,
Class S for Floor Beams and
Primary Truss Members, and
Class U for Main Truss Members.
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines
Weight, in Pounds per Linear Foot of Bridge
30 000
28 000
26 000
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8 000
6 000
4 000
2 000
300
450
600
750
900
1 050
1 200
1 350
1 500
1 650
1 800
Length of Main Opening, in Feet

Fig. 19.

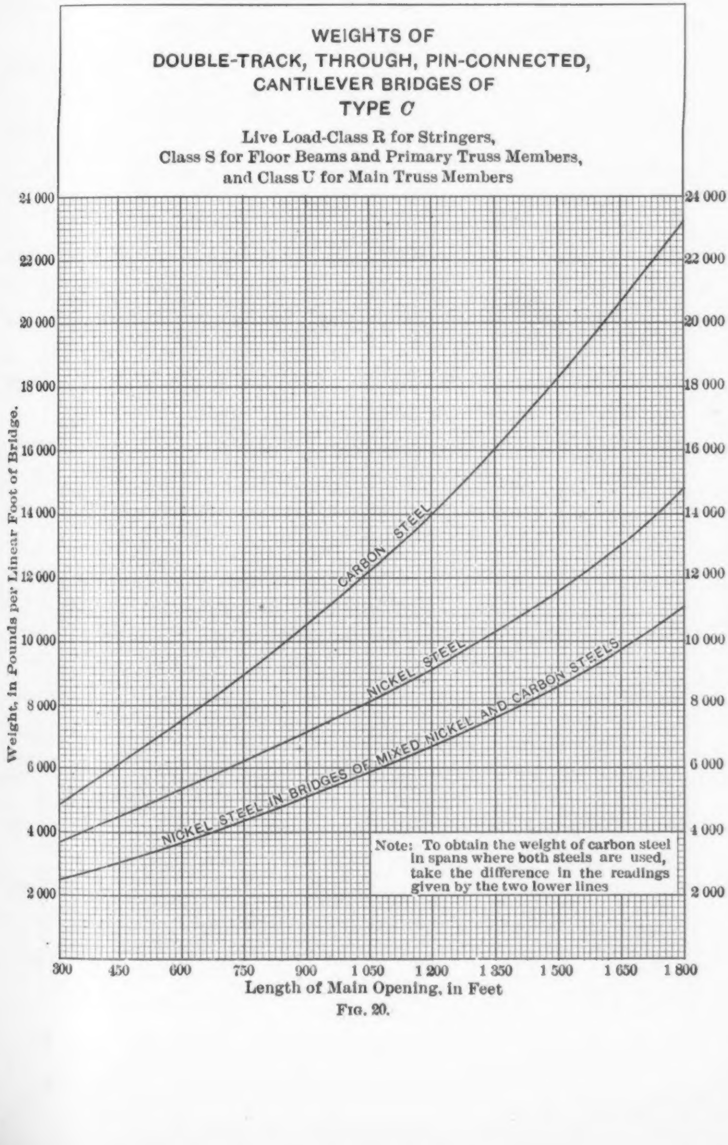

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
CANTILEVER BRIDGES OF
TYPE C
Live Load-Class R for Stringers,
Class S for Floor Beams and Primary Truss Members,
and Class U for Main Truss Members
Weight, in Pounds per Linear Foot of Bridge.
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8 000
6 000
4 000
2 000
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN BRIDGES OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines
300
450
600
750
900
1 050
1 200
1 350
1 500
1 650
1 800
Length of Main Opening, in Feet

FIG. 20.

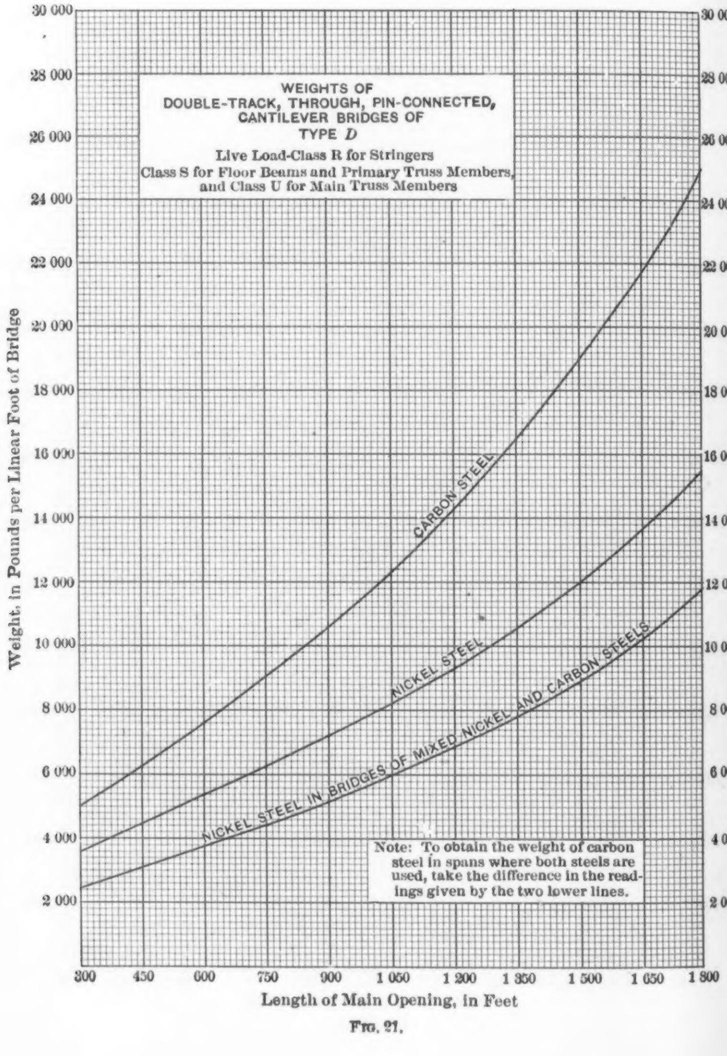

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, CANTILEVER BRIDGES OF TYPE *D*
Live Load-Class R for Stringers
Class S for Floor Beams and Primary Truss Members, and Class U for Main Truss Members
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN BRIDGES OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Bridge
30 000
28 000
26 000
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8 000
6 000
4 000
2 000
300
450
600
750
900
1 050
1 200
1 350
1 500
1 650
1 800
Length of Main Opening, in Feet

FIG. 21.

DOUBLE-TRACK RAILROAD CANTILEVER BRIDGES.
TYPICAL LAY-OUTS.

Trusses are spaced to agree with requirements of "De Pontibus."
Loading is Class R for stringers, Class S for floor beams and primary truss members, and Class U for main truss members.
Suspended spans are $\frac{3}{8}$ of main opening and cantilever and anchor arms are each $\frac{5}{16}$ of main opening.
l equals sum of one suspended span and two cantilever arms.

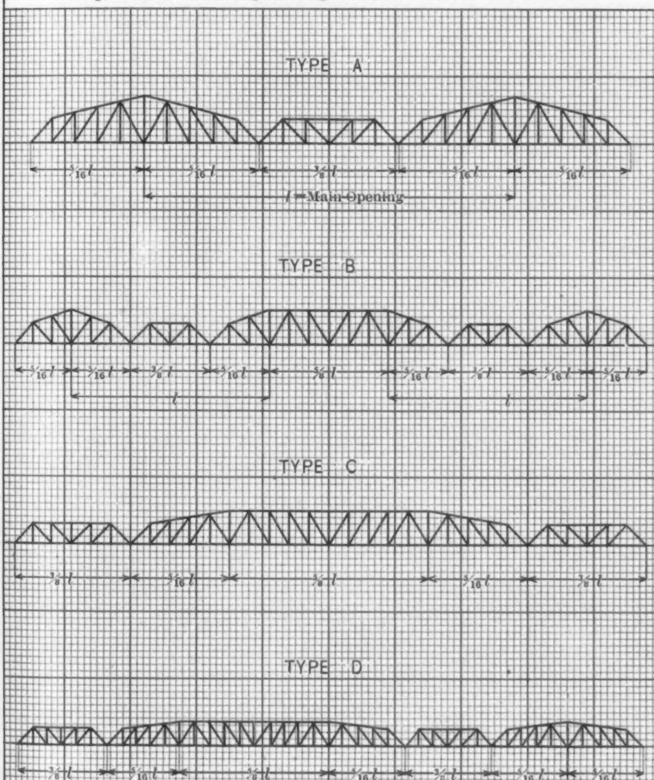
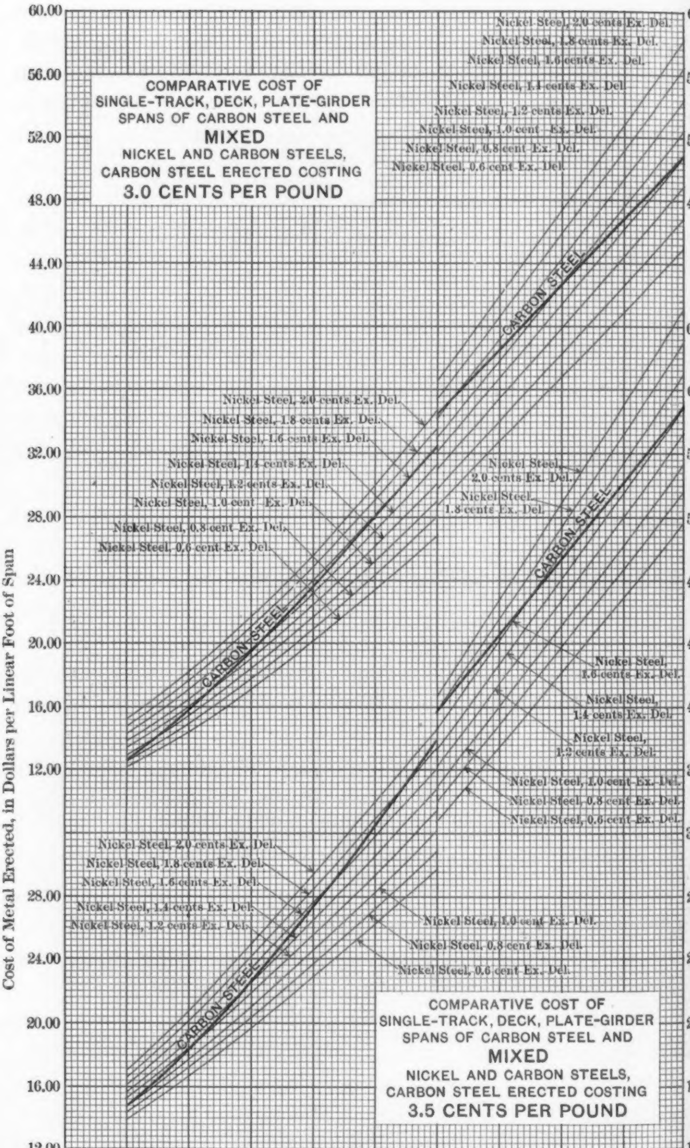


Fig. 22.



FIGS. 23 AND 24.

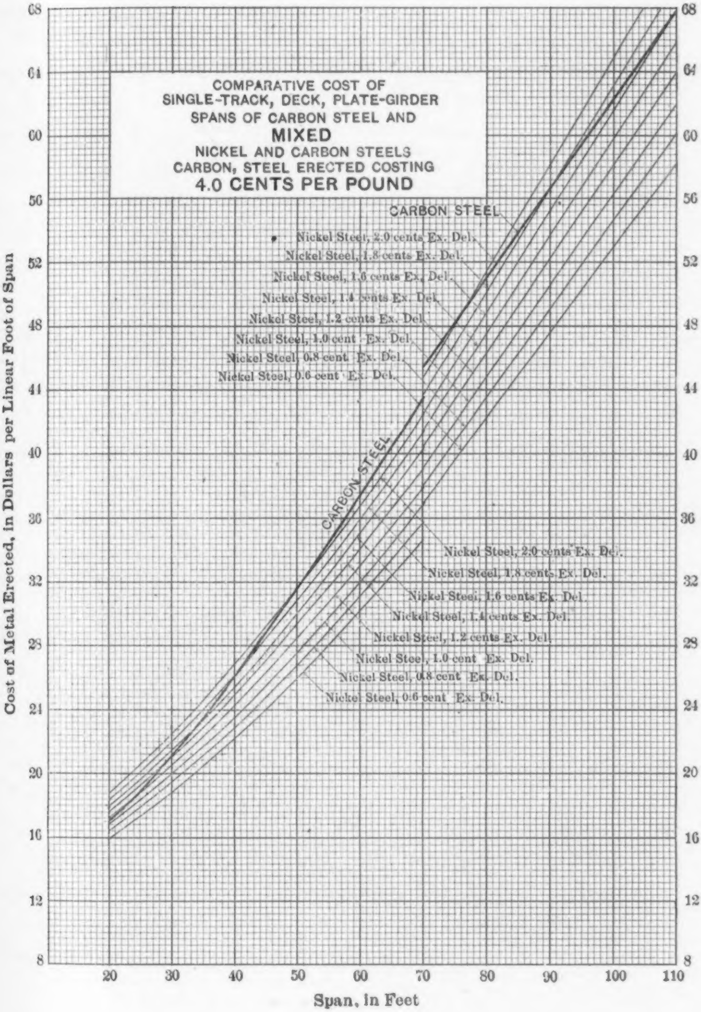
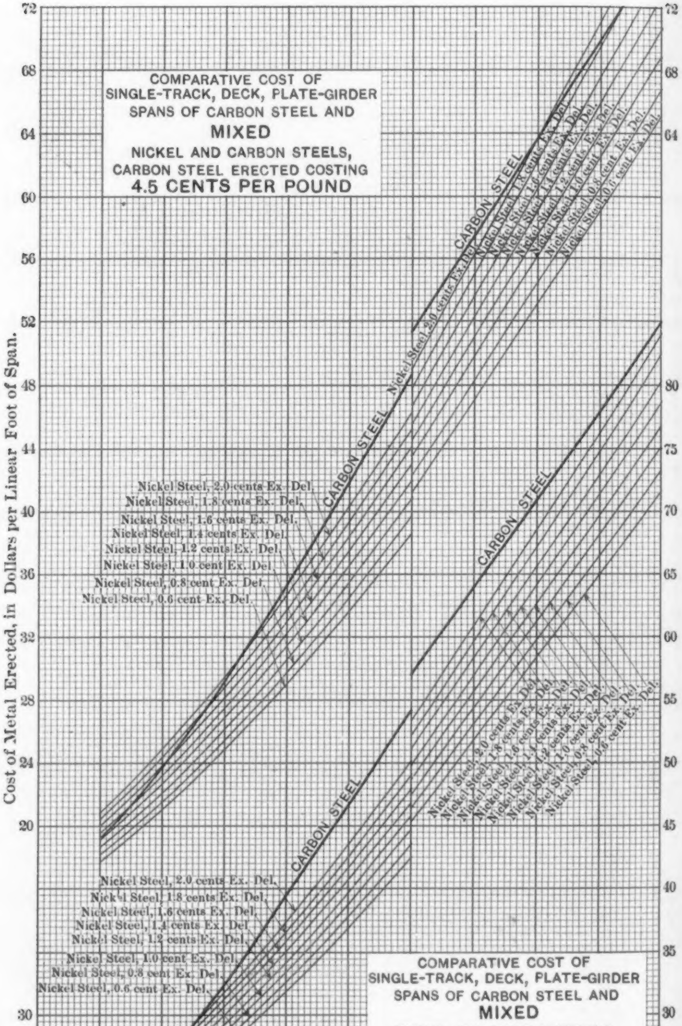

COMPARATIVE COST OF
SINGLE-TRACK, DECK, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS
CARBON, STEEL ERECTED COSTING
4.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
68
64
60
56
52
48
44
40
36
32
28
24
20
16
12
8
20
30
40
50
60
70
80
90
100
110
Span, in Feet

FIG. 25.



FIGS. 26 AND 27.

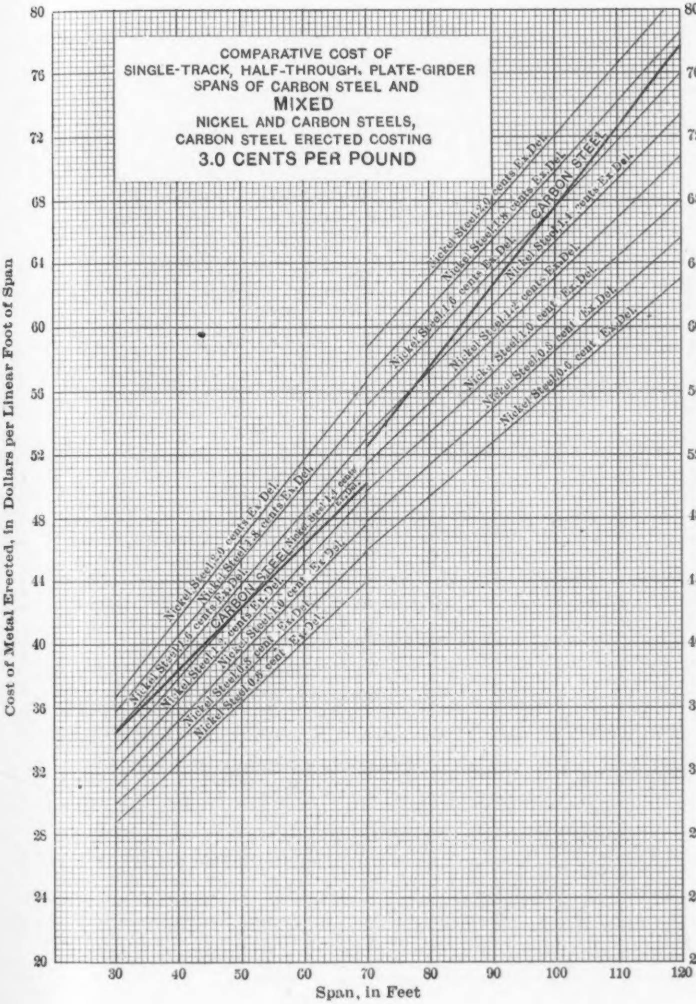

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.0 CENTS PER POUND
Nickel Steel 2.0 cents Ex.Del.
Nickel Steel 1.8 cents Ex.Del.
Nickel Steel 1.6 cents Ex.Del.
CARBON STEEL
Nickel Steel 1.4 cents Ex.Del.
Nickel Steel 1.2 cents Ex.Del.
Nickel Steel 1.0 cent Ex.Del.
Nickel Steel 0.8 cent Ex.Del.
Nickel Steel 0.6 cent Ex.Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
20 24 28 32 36 40 44 48 52 56 60 64 68 72 76 80
30 40 50 60 70 80 90 100 110 120

Fig. 28.

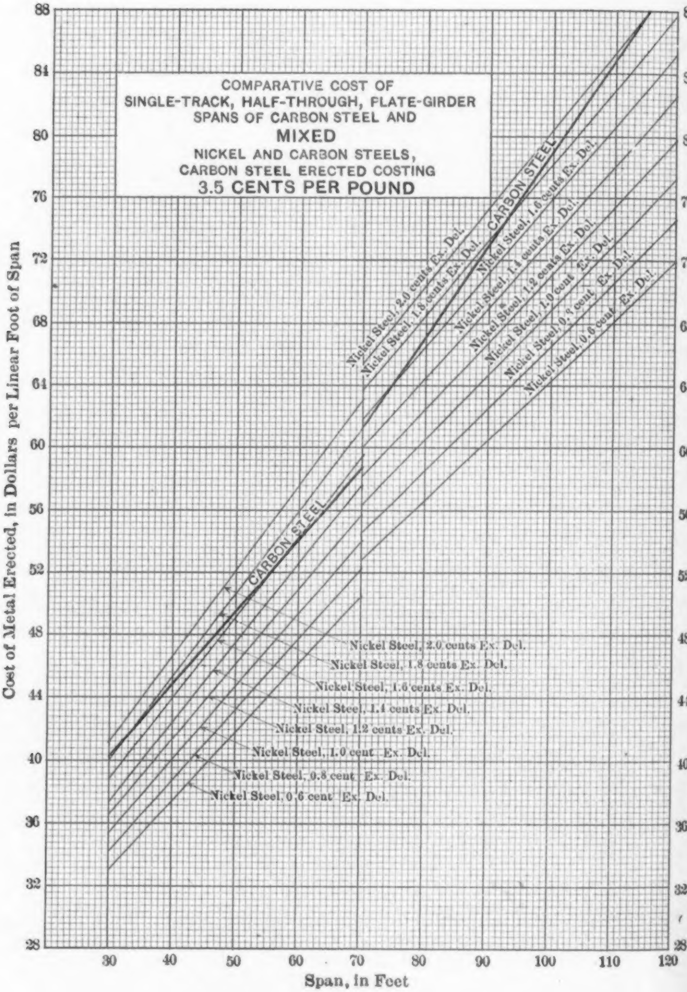

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
28
32
36
40
44
48
52
56
60
64
68
72
76
80
84
88
30
40
50
60
70
80
90
100
110
120

FIG. 29.

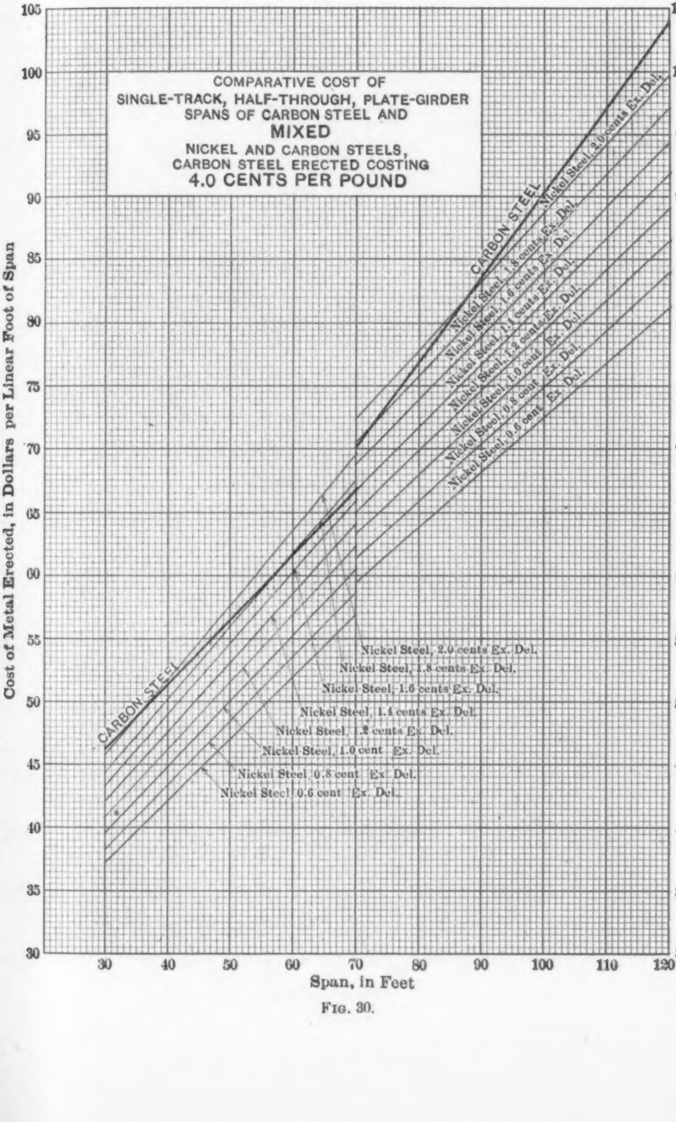

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND
CARBON STEEL
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
105
110
120

FIG. 30.

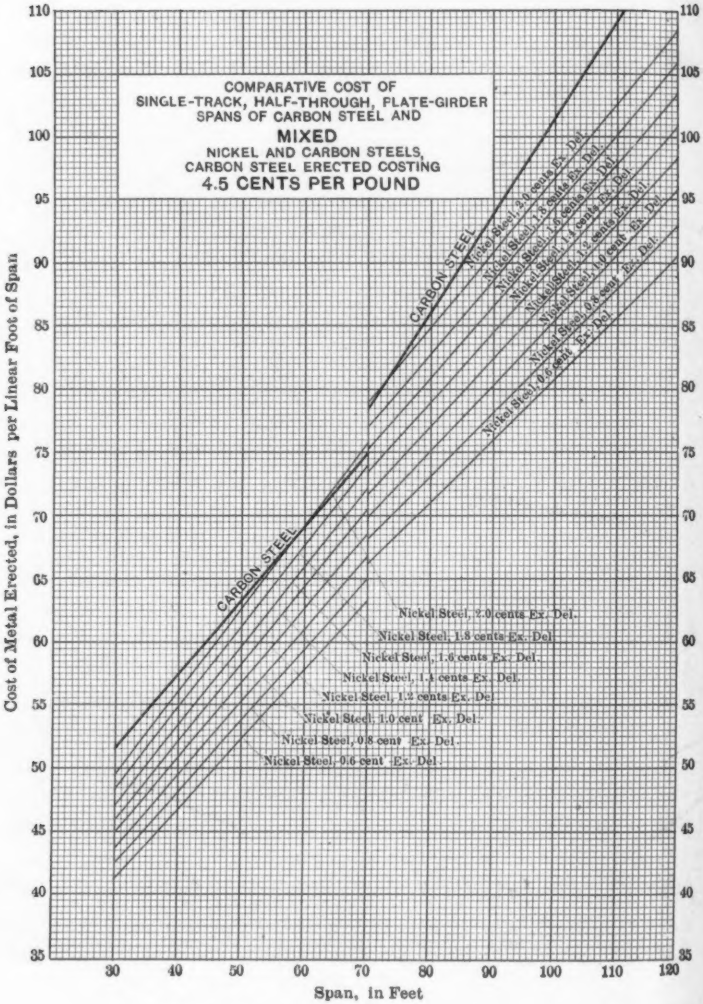

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
35
40
45
50
55
60
65
70
75
80
85
90
95
100
105
110
30
40
50
60
70
80
90
100
110
120
Span, in Feet

Fig. 31.

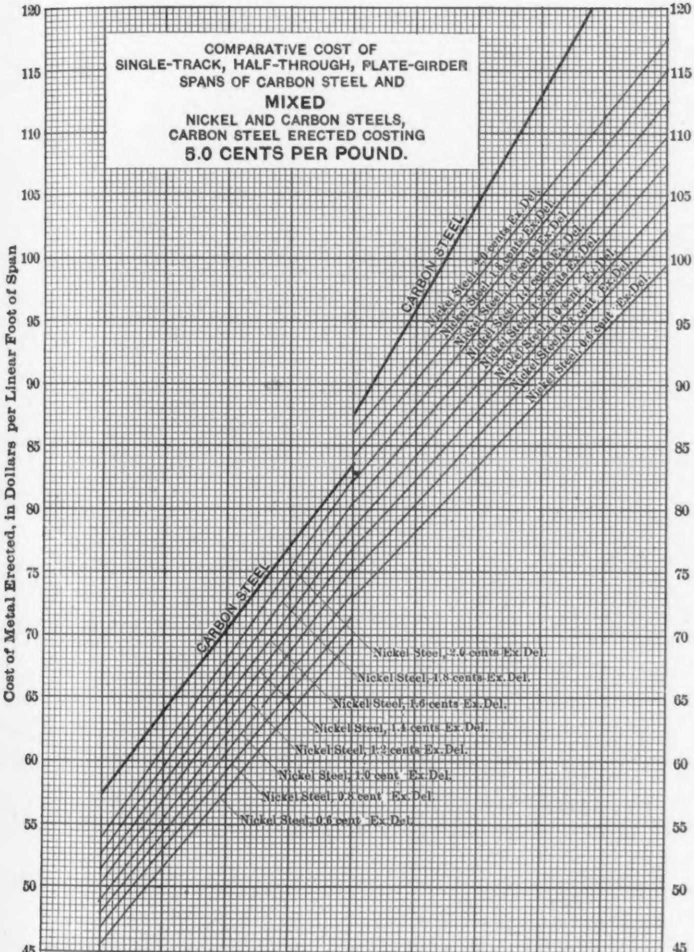
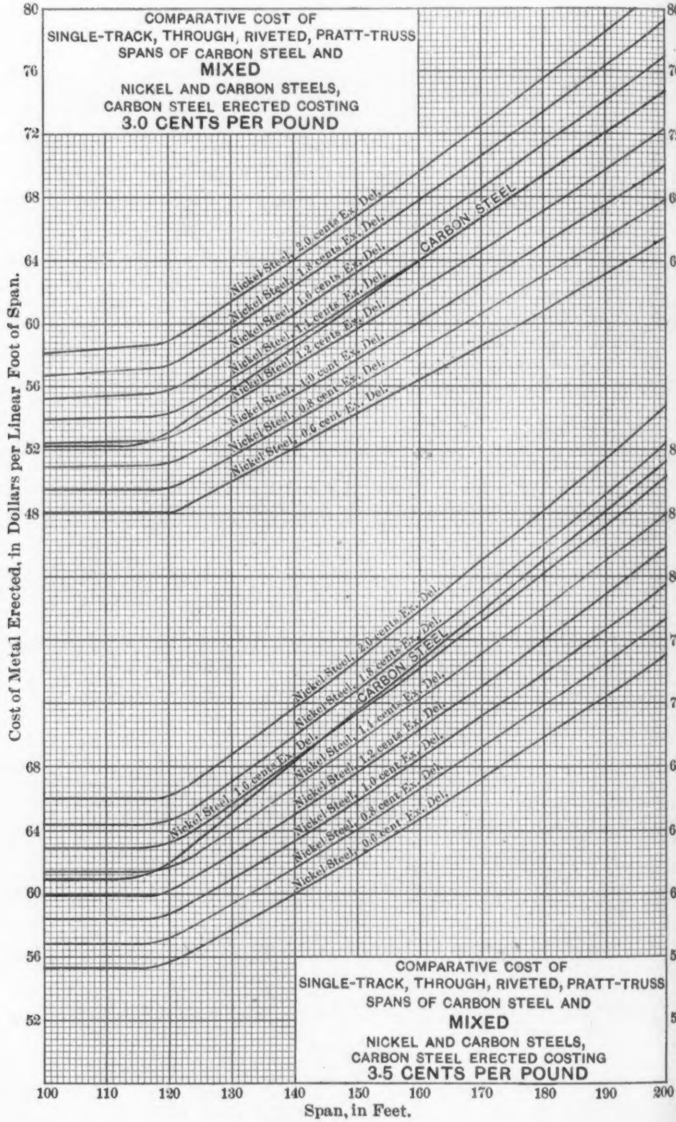
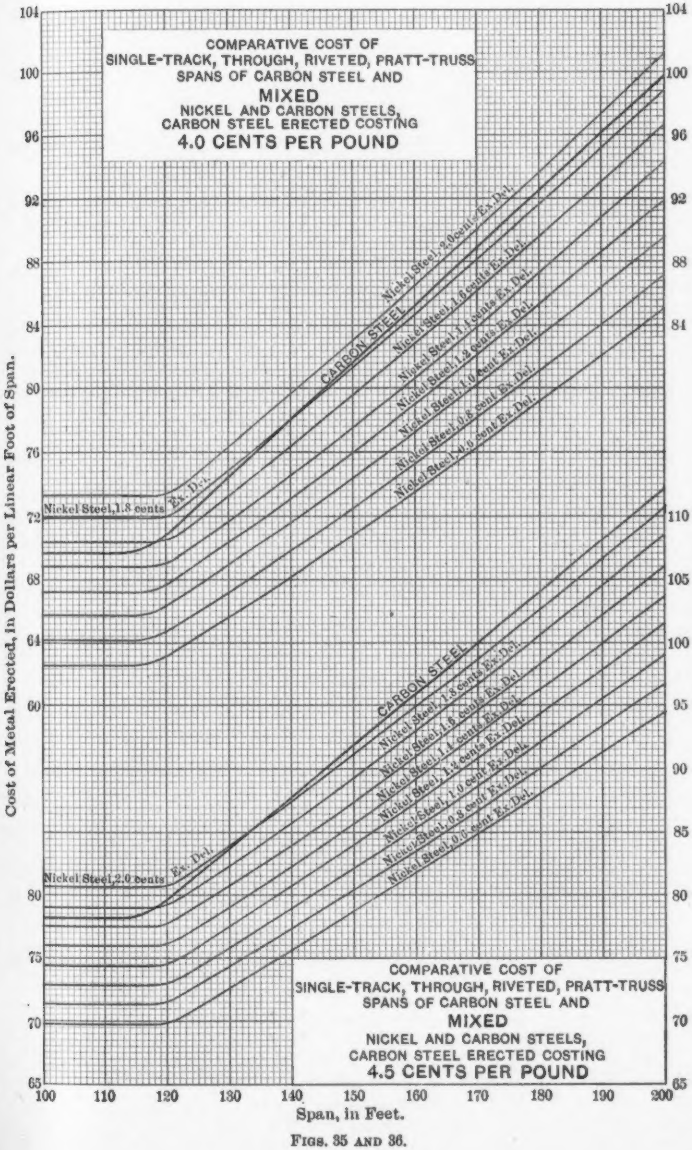

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND.
CARBON STEEL
CARBON STEEL
Nickel Steel, 2.0 cents Ex.Del.
Nickel Steel, 1.8 cents Ex.Del.
Nickel Steel, 1.6 cents Ex.Del.
Nickel Steel, 1.4 cents Ex.Del.
Nickel Steel, 1.2 cents Ex.Del.
Nickel Steel, 1.0 cent Ex.Del.
Nickel Steel, 0.8 cent Ex.Del.
Nickel Steel, 0.6 cent Ex.Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
45
50
55
60
65
70
75
80
85
90
95
100
105
110
115
120
30
40
50
60
70
80
90
100
110
120

FIG. 32.



FIGS. 33 AND 34.



FIGS. 35 AND 36.

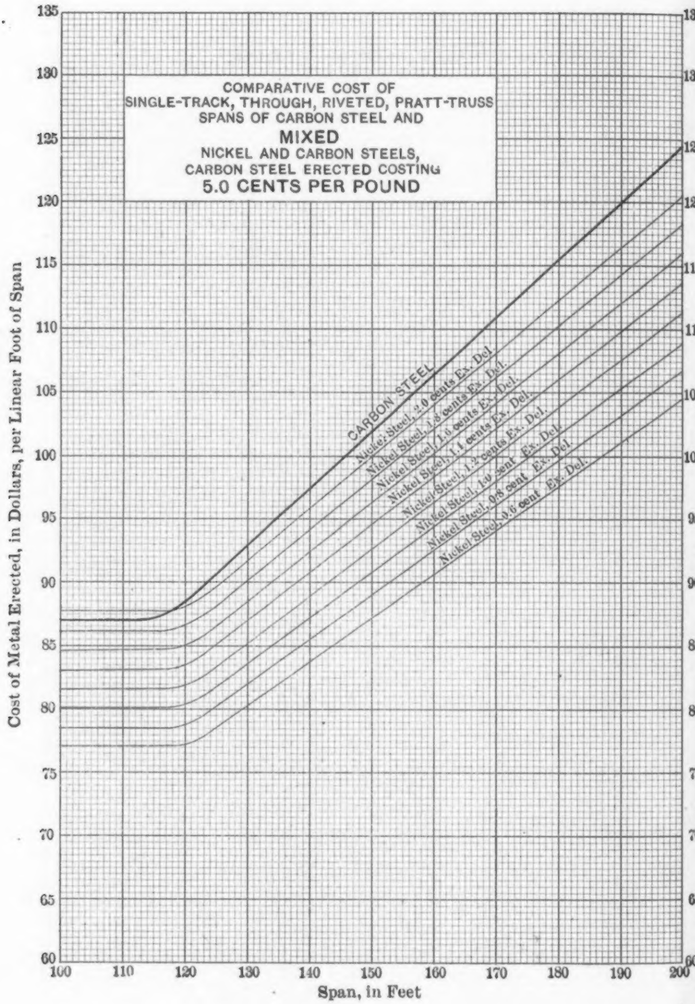
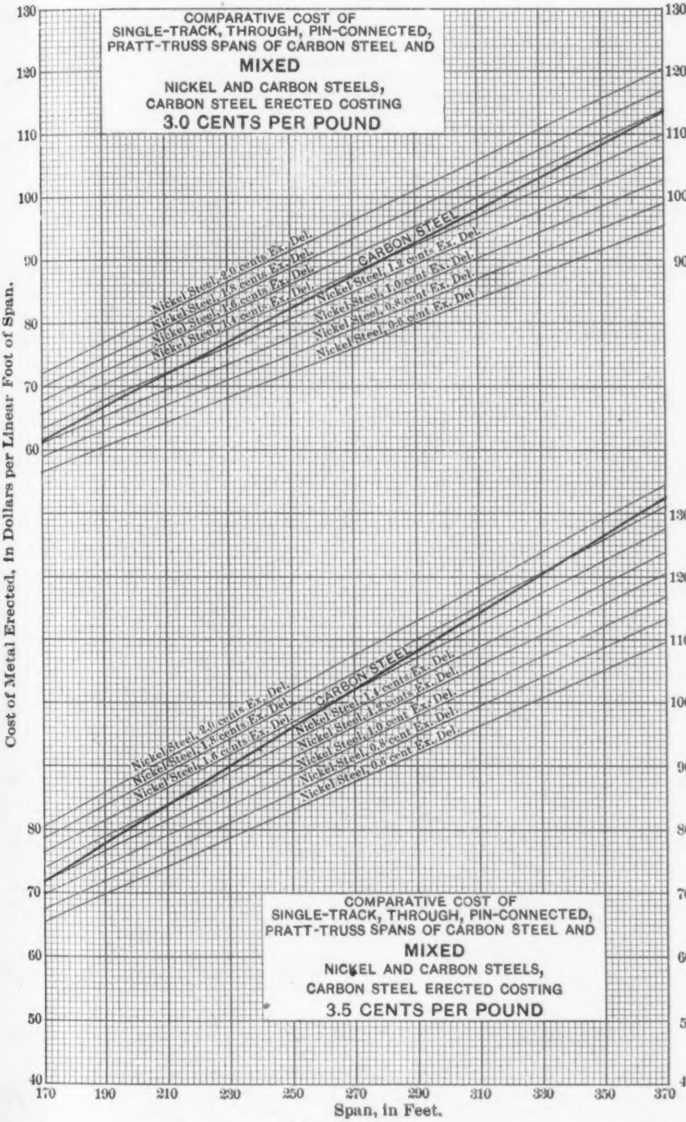
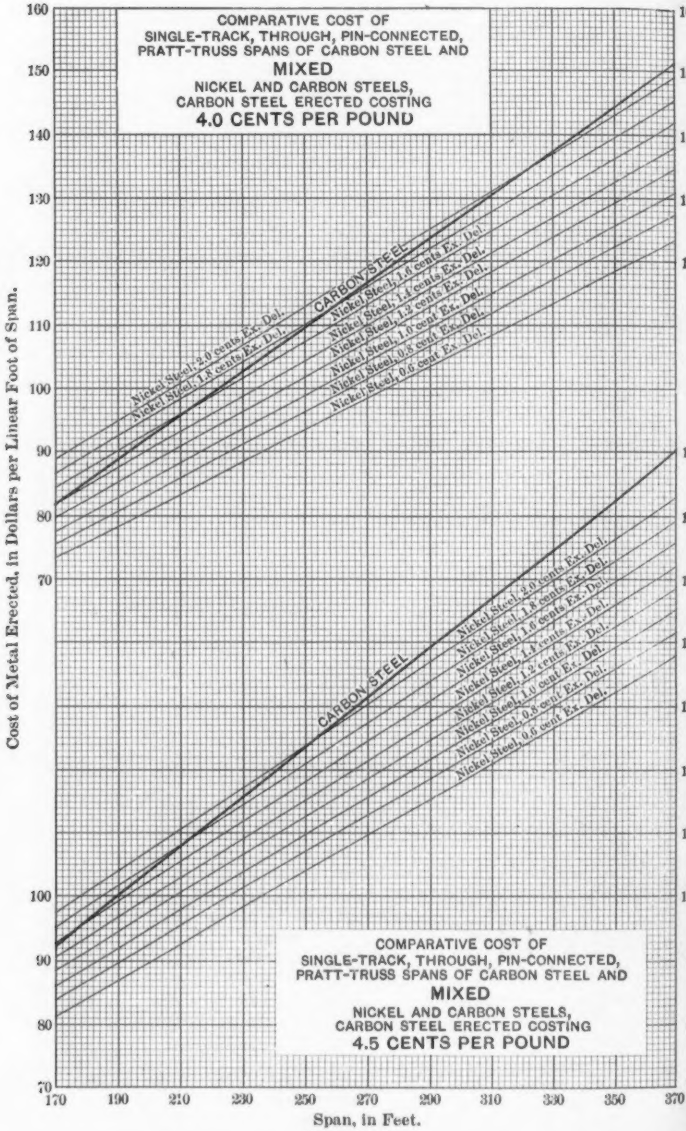

COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.1 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars, per Linear Foot of Span
60 65 70 75 80 85 90 95 100 105 110 115 120 125 130 135
100 110 120 130 140 150 160 170 180 190 200
Span, in Feet

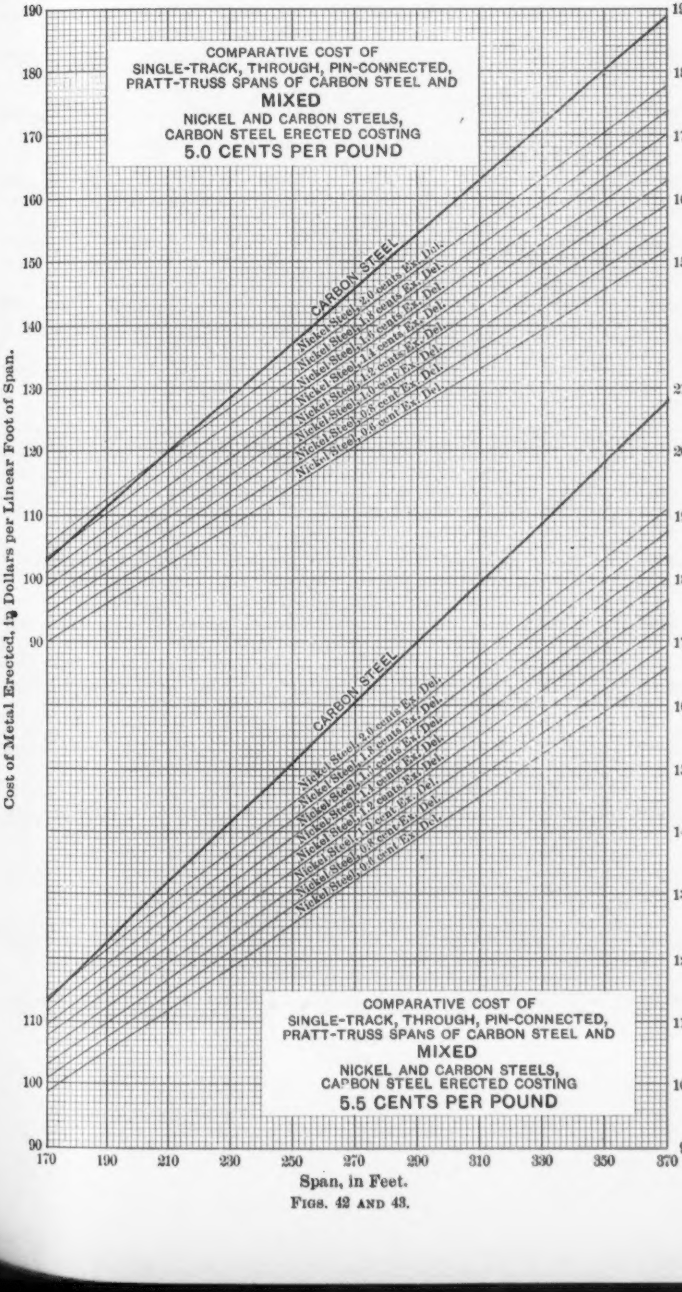
Fig. 37.



Figs. 38 and 39.



FIGS. 40 AND 41.



FIGS. 42 AND 43.

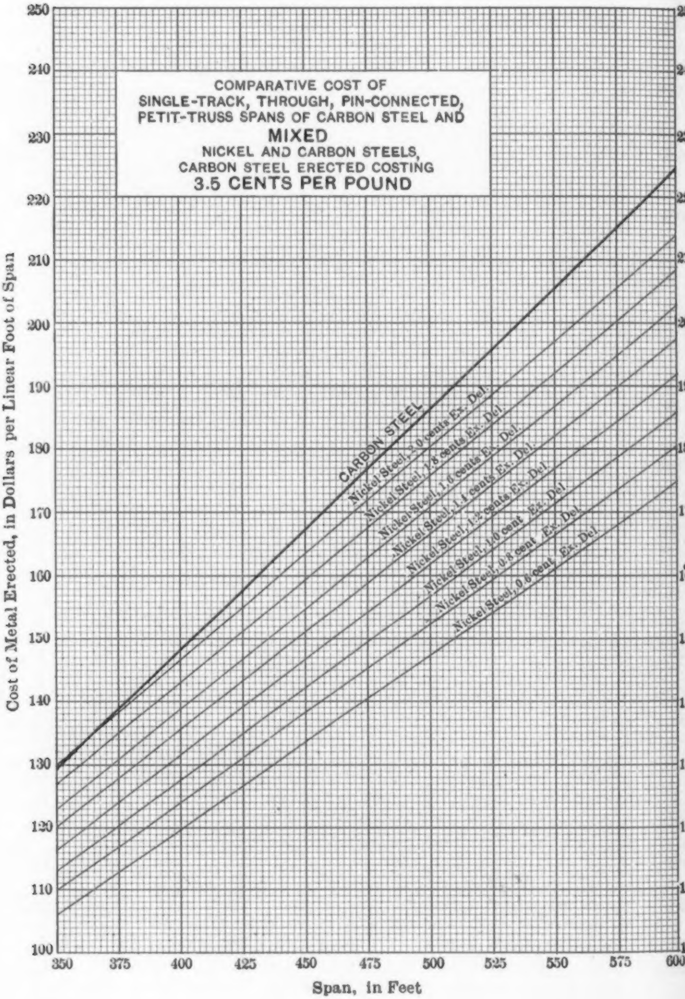

COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet

Fig. 44.

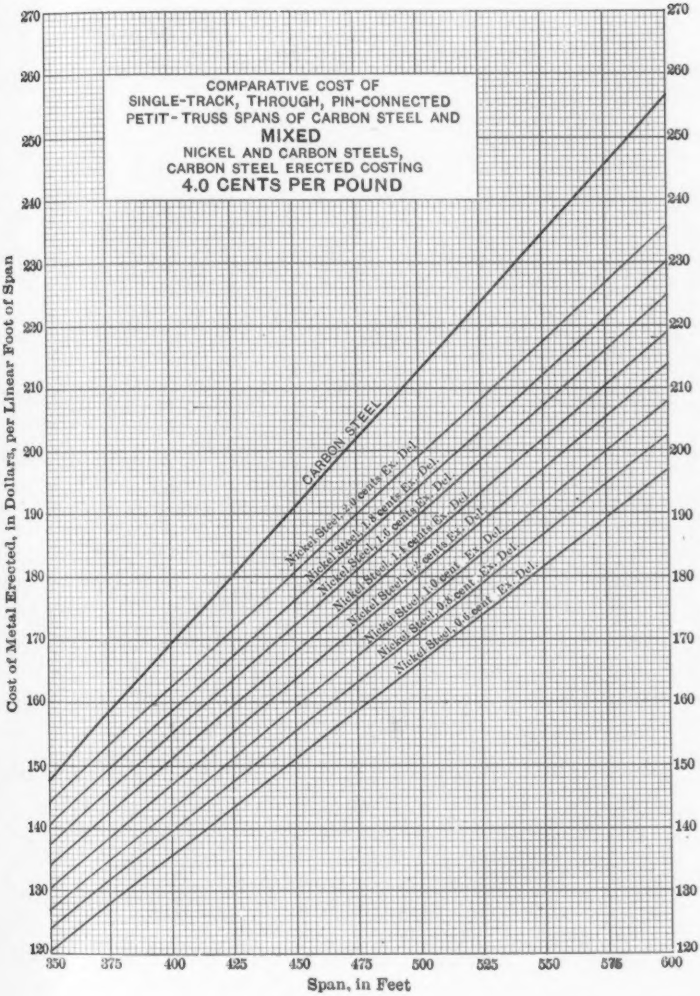
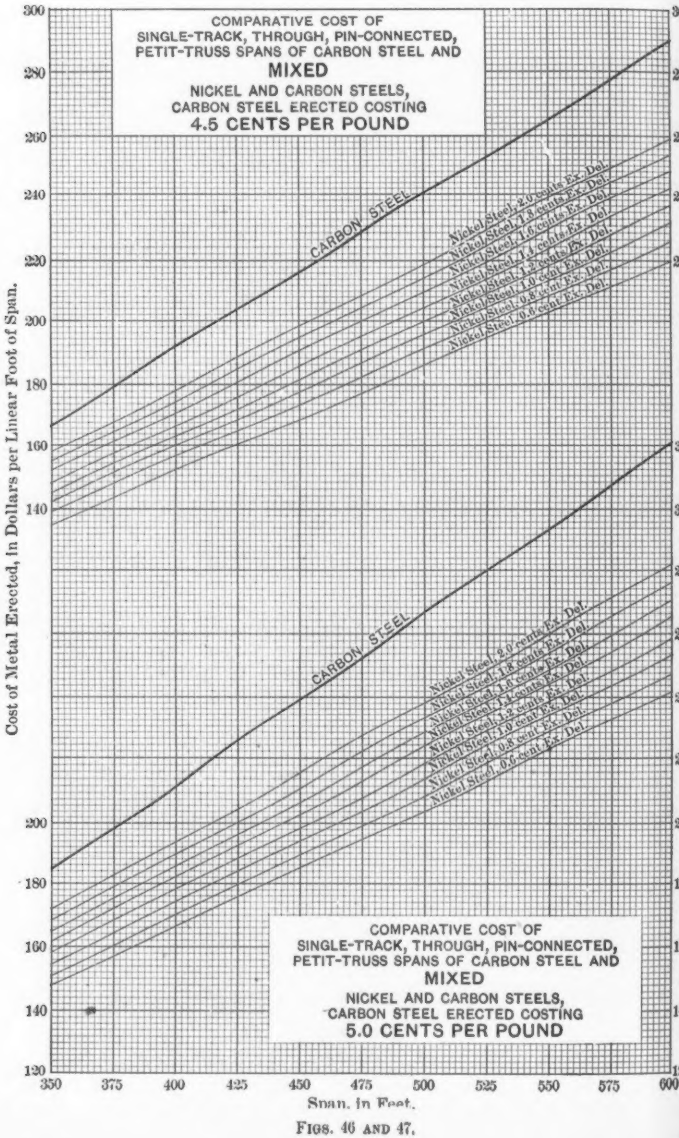

COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars, per Linear Foot of Span
270
260
250
240
230
220
210
200
190
180
170
160
150
140
130
120
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet

FIG. 45.



FIGS. 46 AND 47.

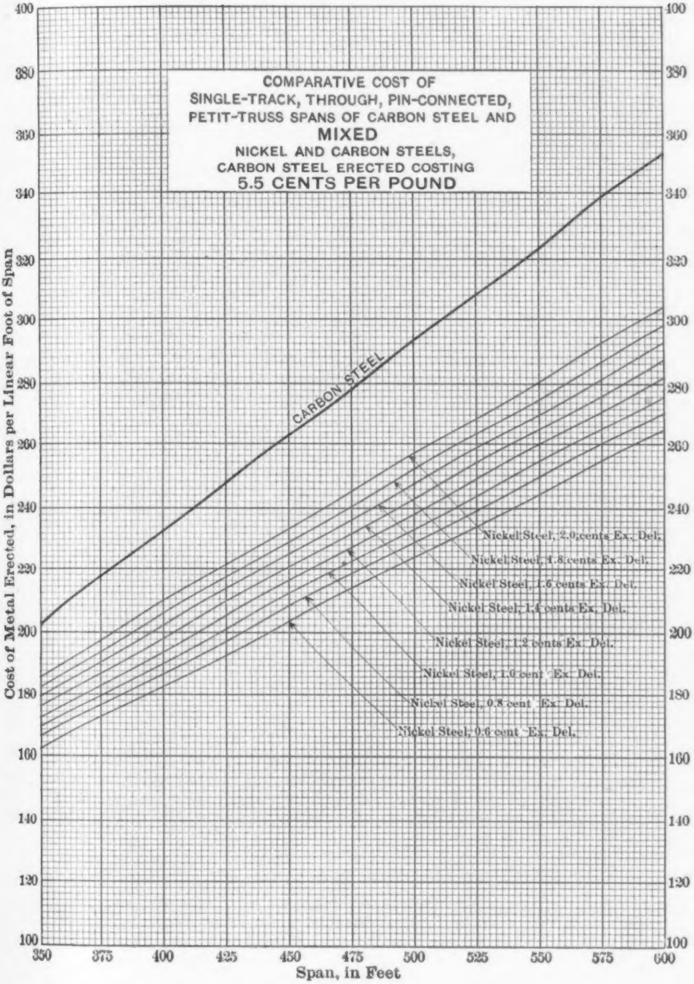
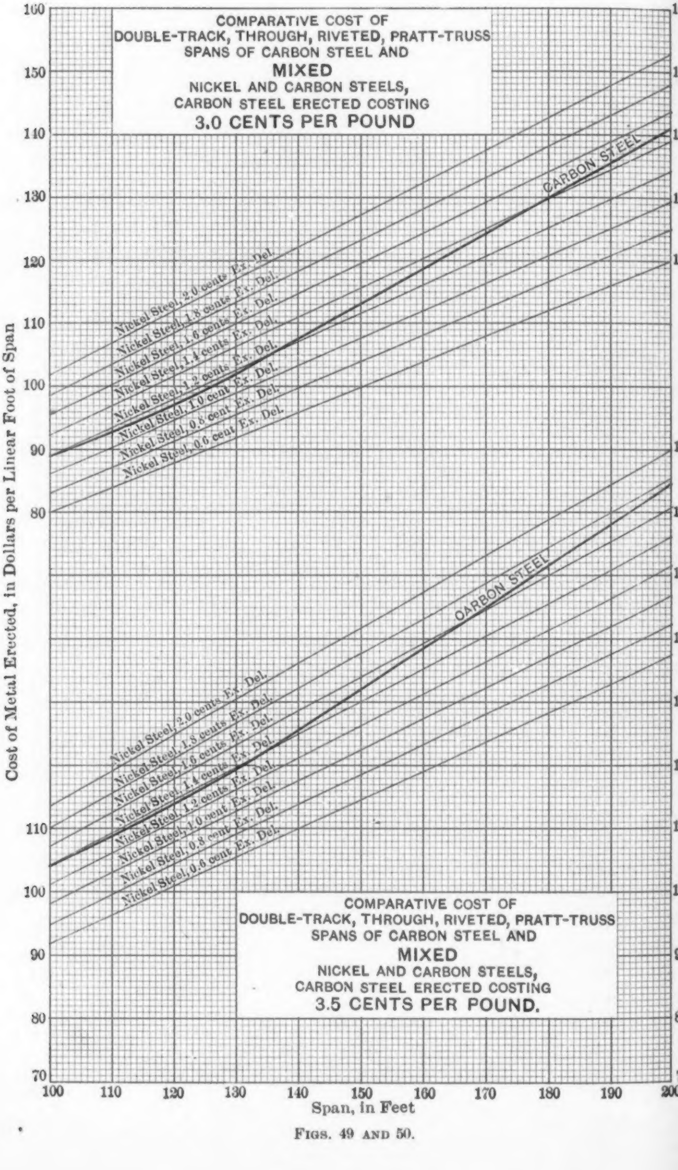
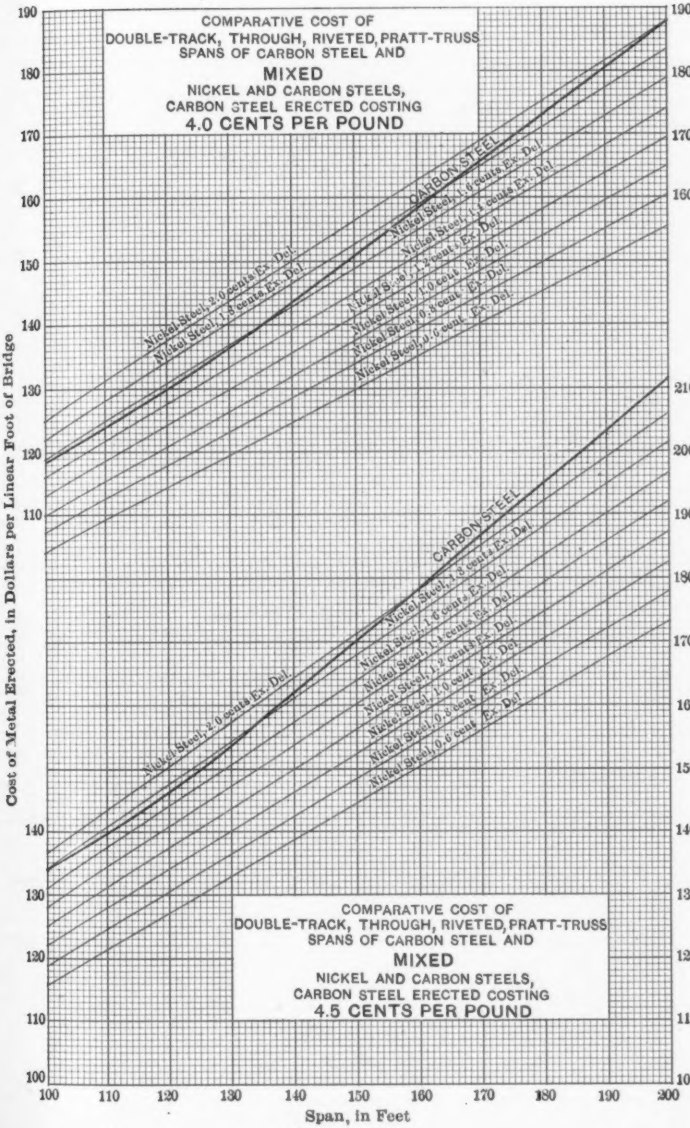

COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
400
380
360
340
320
300
280
260
240
220
200
180
160
140
120
100
350
375
400
425
450
475
500
525
550
575
600

Fig. 48.



FIGS. 49 AND 50.



FIGS. 51 AND 52.

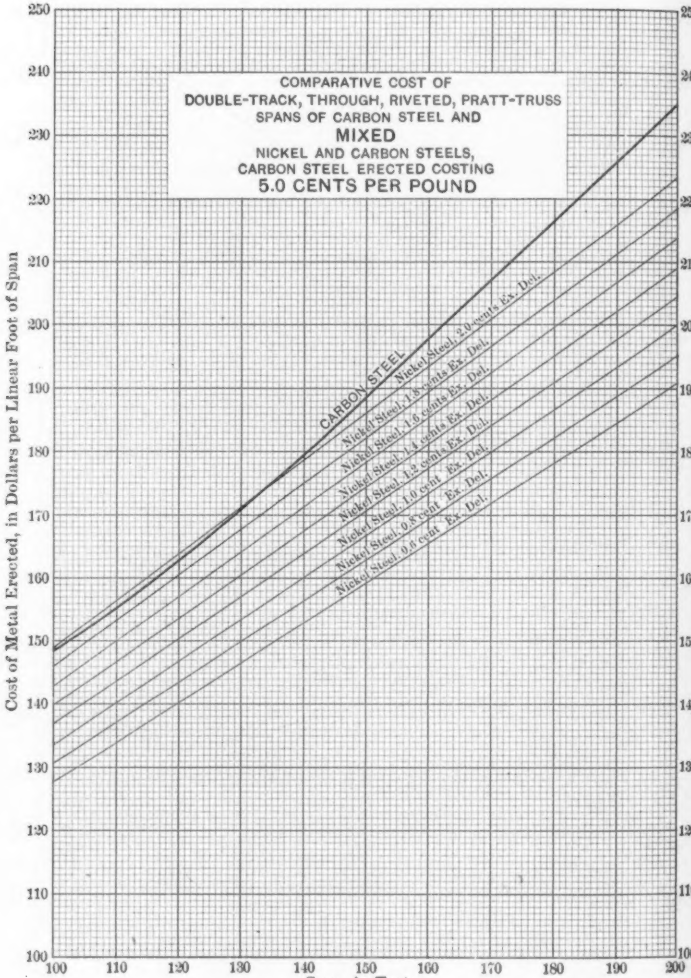

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
100 110 120 130 140 150 160 170 180 190 200
100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250

Fig. 53.

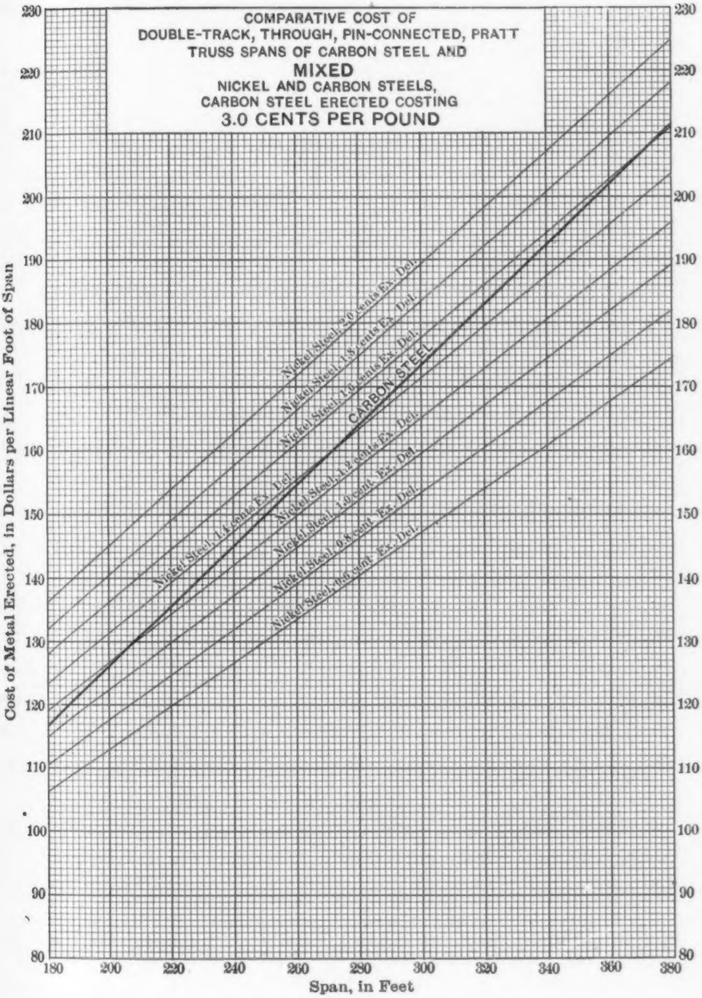

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, PRATT
TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.0 CENTS PER POUND
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
CARBON STEEL
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
180
200
220
240
260
280
300
320
340
360
380

Fig. 54.

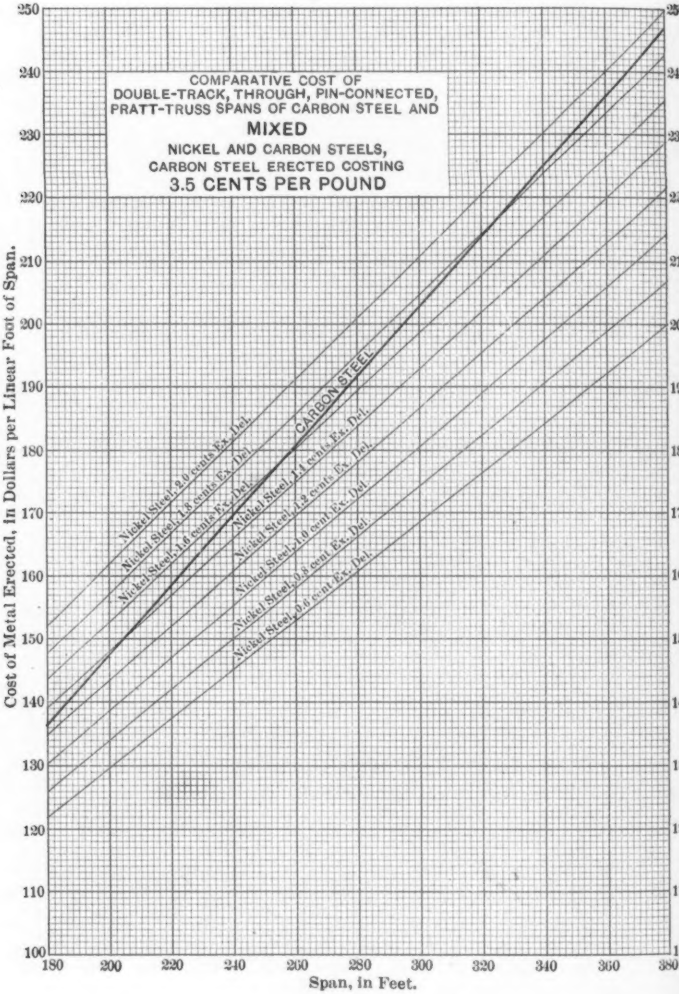

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span.
Span, in Feet.
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
180
200
220
240
260
280
300
320
340
360
380

FIG. 55.

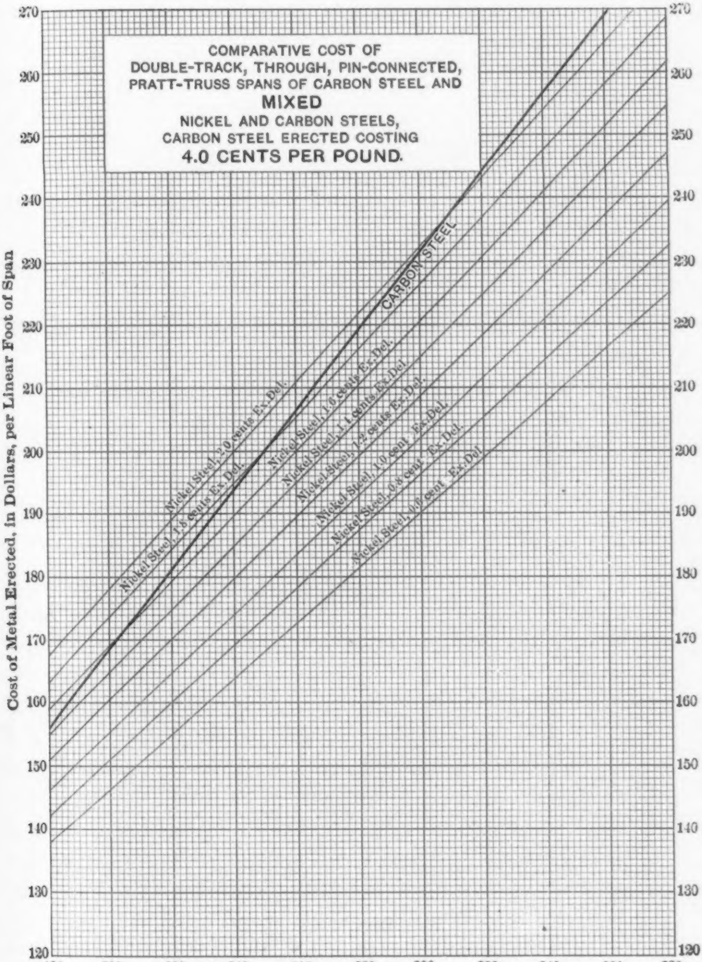

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND.
CARBON STEEL
Nickel Steel, 2.0 cents Ex.Del.
Nickel Steel, 1.8 cents Ex.Del.
Nickel Steel, 1.6 cents Ex.Del.
Nickel Steel, 1.4 cents Ex.Del.
Nickel Steel, 1.2 cents Ex.Del.
Nickel Steel, 1.0 cent Ex.Del.
Nickel Steel, 0.8 cent Ex.Del.
Nickel Steel, 0.6 cent Ex.Del.
Cost of Metal Erected, in Dollars, per Linear Foot of Span
Span, in Feet
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
180
200
220
240
260
280
300
320
340
360
380

FIG. 56.

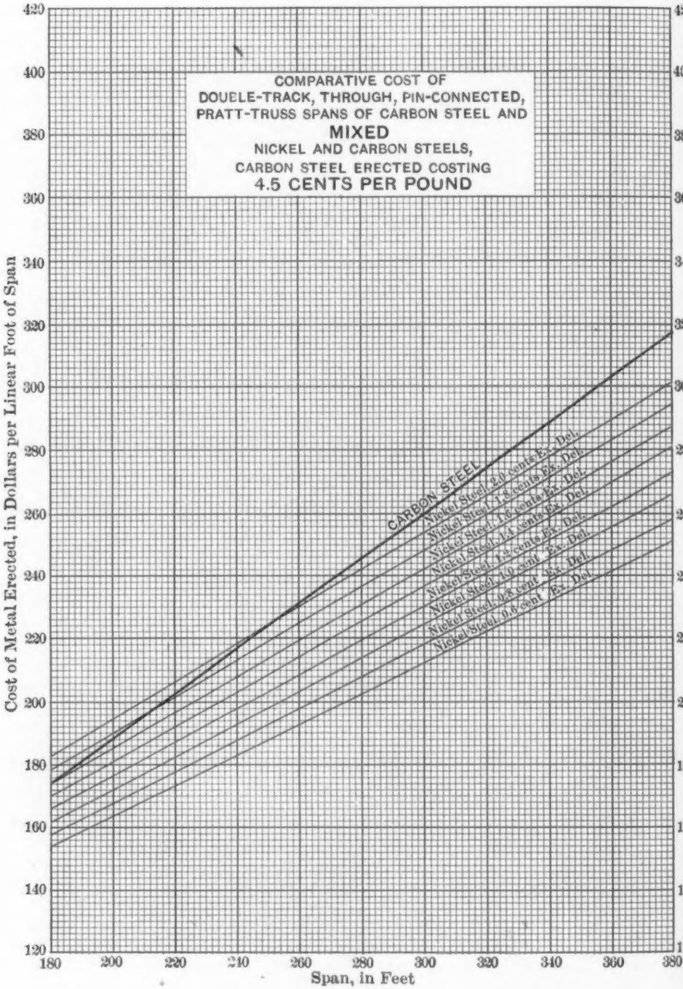
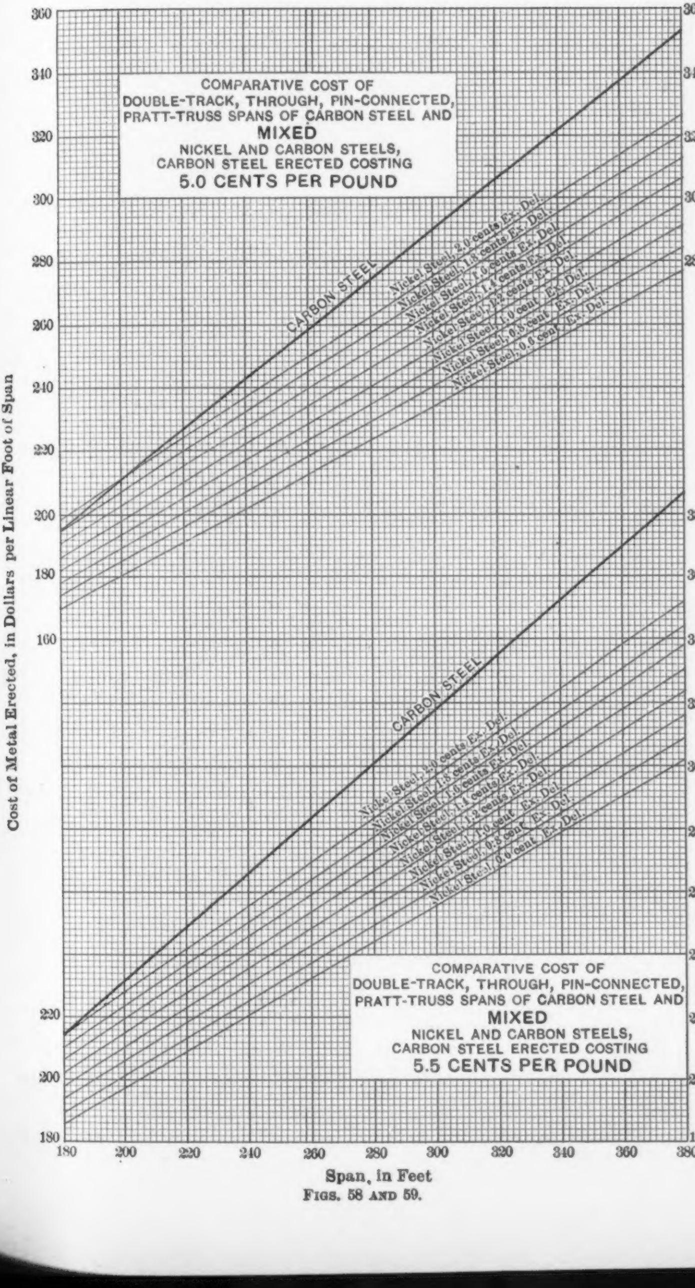
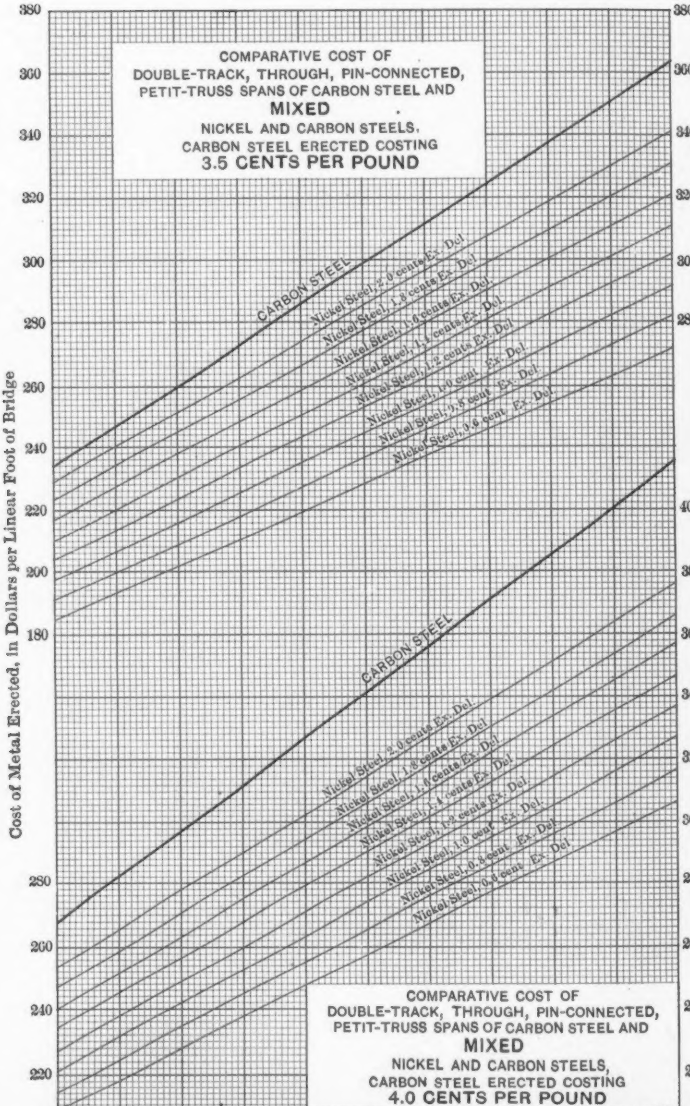

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
180
200
220
240
260
280
300
320
340
360
380
Span, in Feet

FIG. 57.



FIGS. 58 AND 59.



FIGS. 60 AND 61.

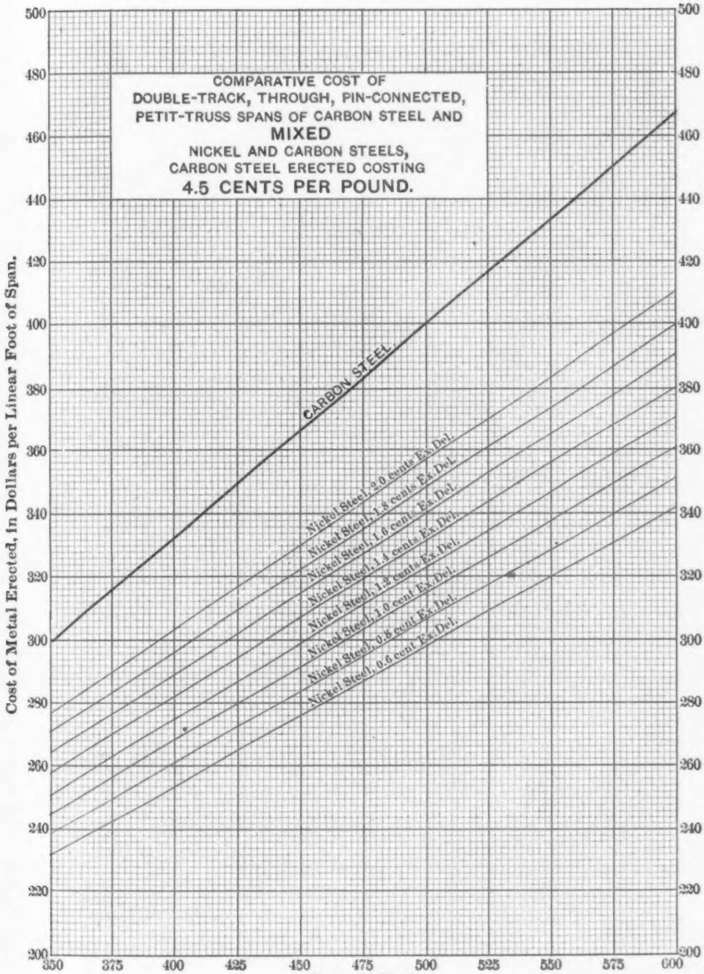

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND.
CARBON STEEL
Nickel Steel, 2.0 cents Ex.Del.
Nickel Steel, 1.8 cents Ex.Del.
Nickel Steel, 1.6 cents Ex.Del.
Nickel Steel, 1.4 cents Ex.Del.
Nickel Steel, 1.2 cents Ex.Del.
Nickel Steel, 1.0 cent Ex.Del.
Nickel Steel, 0.8 cent Ex.Del.
Nickel Steel, 0.6 cent Ex.Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span.
Span, in Feet
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
350
375
400
425
450
475
500
525
550
575
600

FIG. 62.

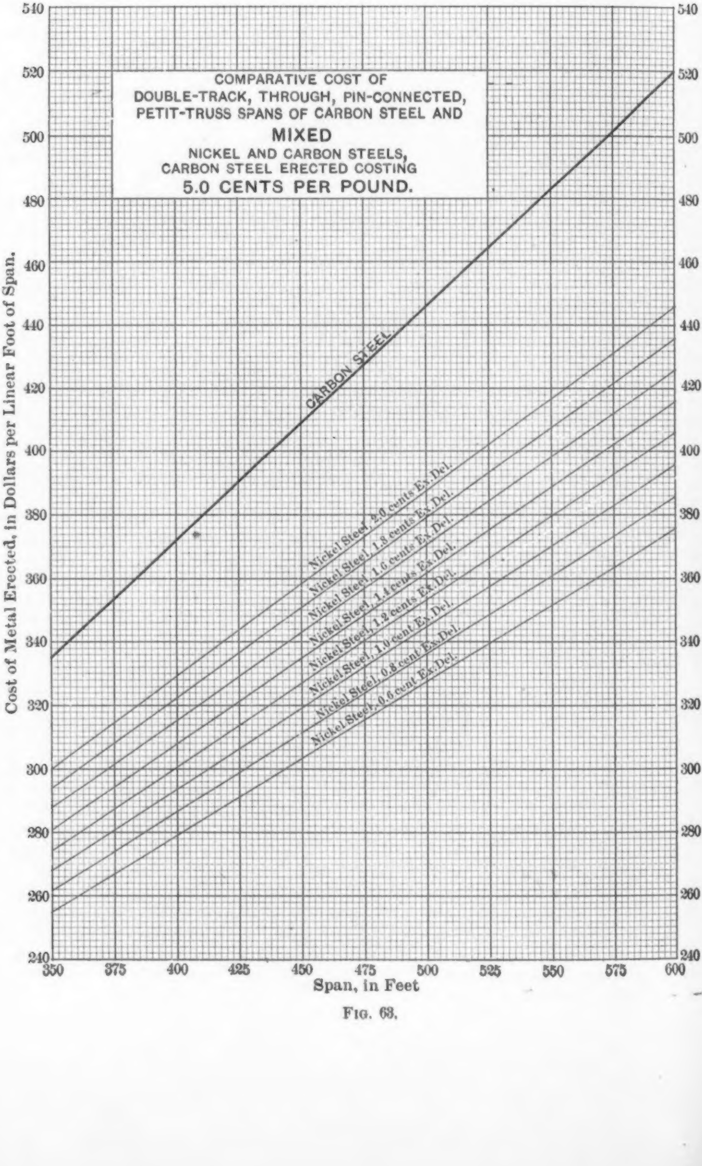

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND.
CARBON STEEL
Nickel Steel, 2.0 cents Ex.Del.
Nickel Steel, 1.8 cents Ex.Del.
Nickel Steel, 1.6 cents Ex.Del.
Nickel Steel, 1.4 cents Ex.Del.
Nickel Steel, 1.2 cents Ex.Del.
Nickel Steel, 1.0 cent Ex.Del.
Nickel Steel, 0.8 cent Ex.Del.
Nickel Steel, 0.6 cent Ex.Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span.
240
260
280
300
320
340
360
380
400
420
440
460
480
500
520
540
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet

Fig. 63.

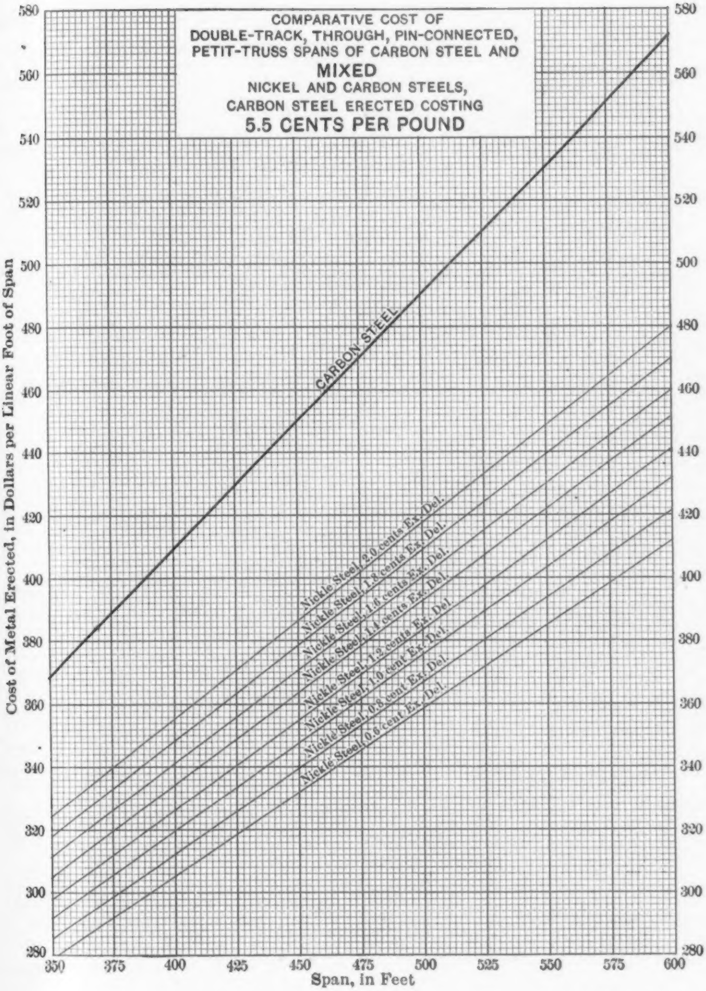
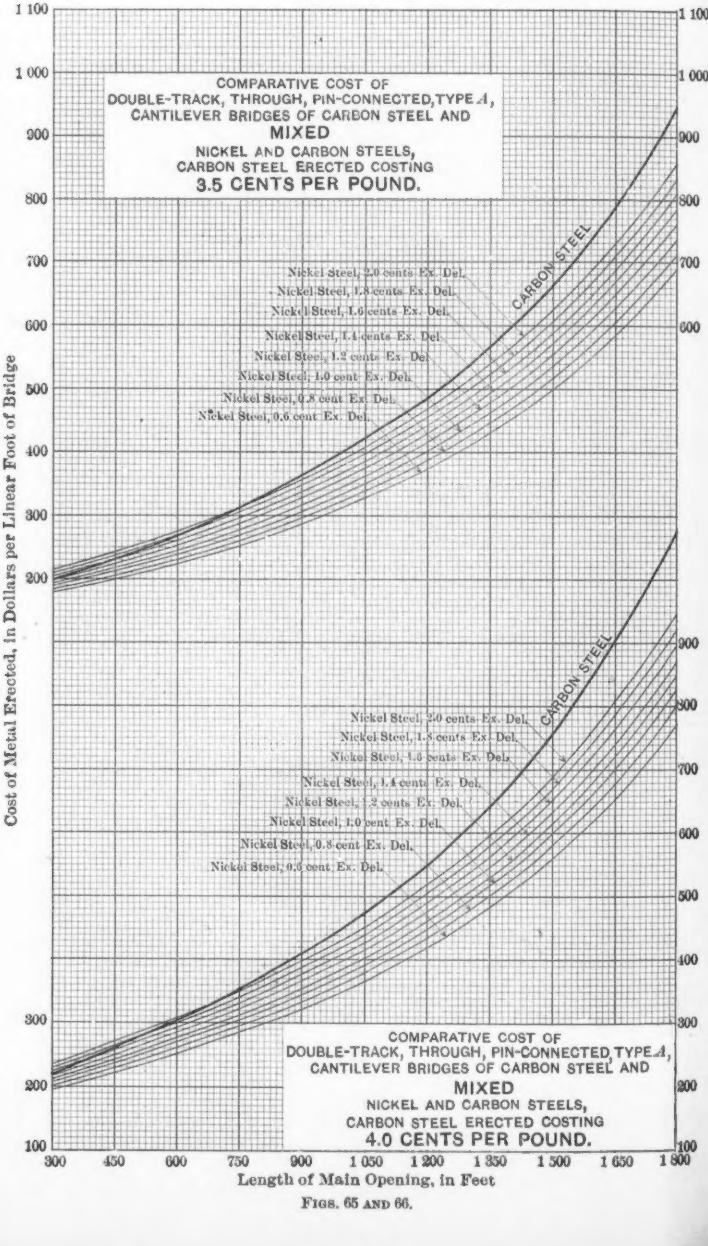
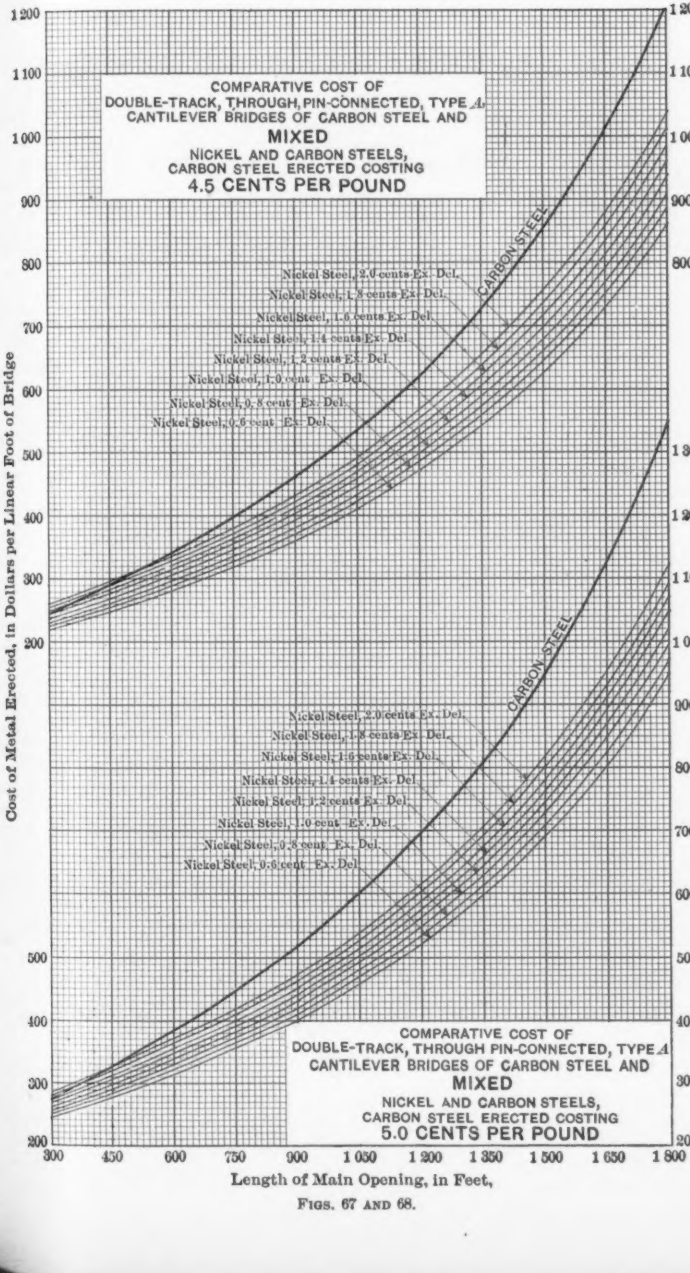

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.5 CENTS PER POUND
CARBON STEEL
Nickle Steel, 2.0 cents Ex. Del.
Nickle Steel, 1.8 cents Ex. Del.
Nickle Steel, 1.6 cents Ex. Del.
Nickle Steel, 1.4 cents Ex. Del.
Nickle Steel, 1.2 cents Ex. Del.
Nickle Steel, 1.0 cent Ex. Del.
Nickle Steel, 0.8 cent Ex. Del.
Nickle Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
580
560
540
520
500
480
460
440
420
400
380
360
340
320
300
280
350
375
400
425
450
475
500
525
550
575
600

FIG. 64.



FIGS. 65 AND 66.



FIGS. 67 AND 68.

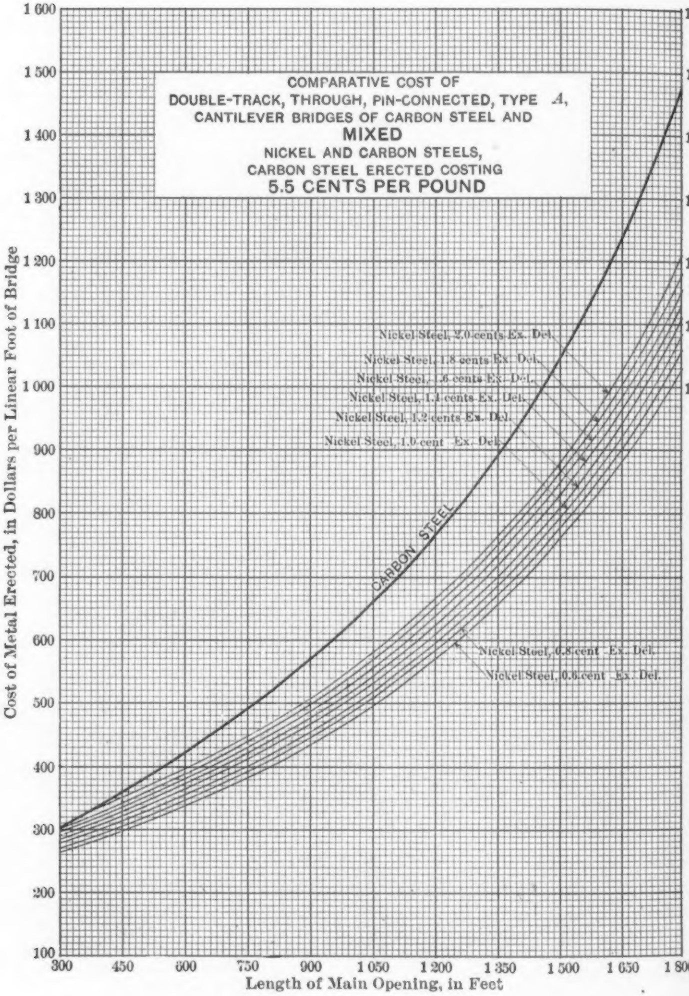

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, TYPE A,
CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Bridge
1 600
1 500
1 400
1 300
1 200
1 100
1 000
900
800
700
600
500
400
300
200
100
300
450
600
750
900
1 050
1 200
1 350
1 500
1 650
1 800
Length of Main Opening, in Feet

FIG. 69.

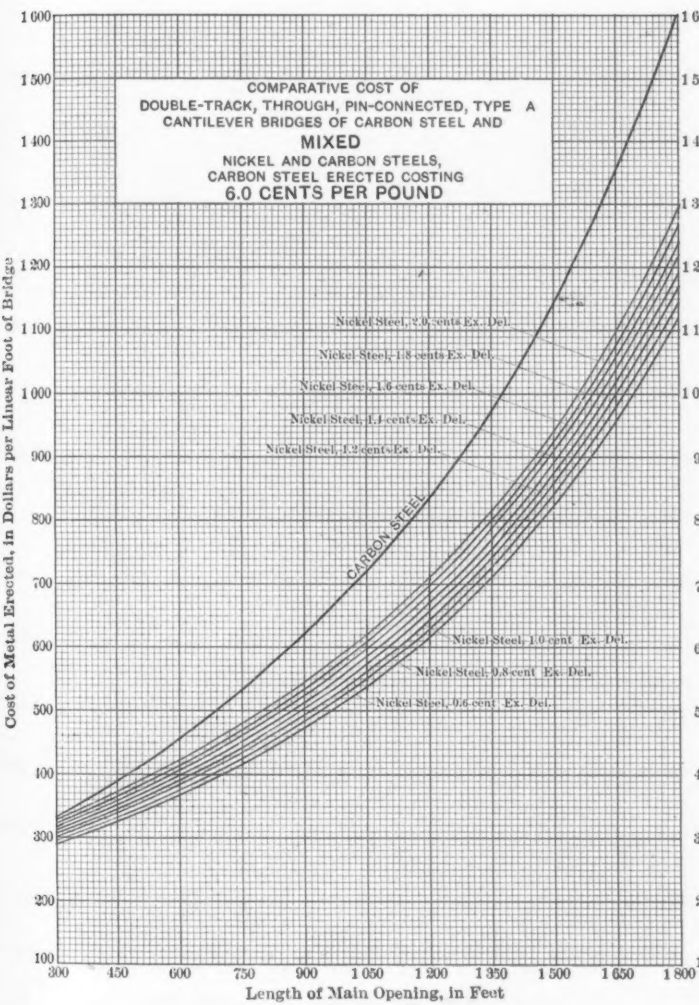

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, TYPE A
CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
6.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Bridge
Length of Main Opening, in Feet

FIG. 70.

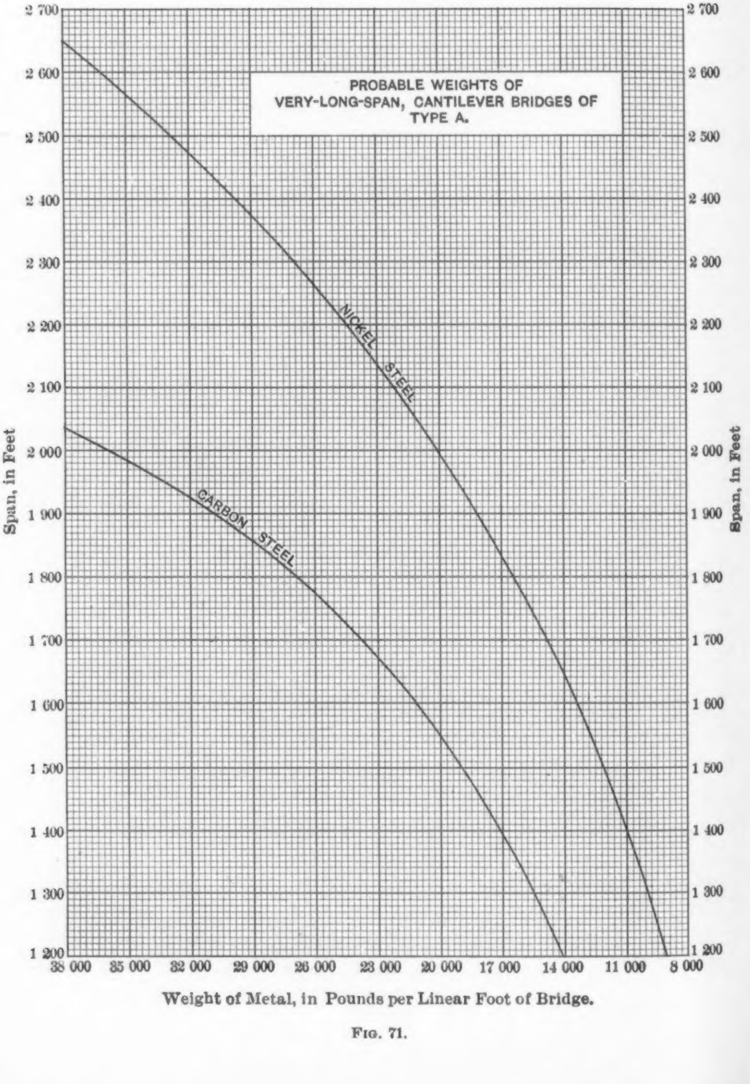

PROBABLE WEIGHTS OF
VERY-LONG-SPAN, CANTILEVER BRIDGES OF
TYPE A.
NICKEL STEEL
CARBON STEEL
Span, in Feet
2 700
2 600
2 500
2 400
2 300
2 200
2 100
2 000
1 900
1 800
1 700
1 600
1 500
1 400
1 300
1 200
38 000
35 000
32 000
29 000
26 000
23 000
20 000
17 000
14 000
11 000
8 000
Weight of Metal, in Pounds per Linear Foot of Bridge.

Fig. 71.

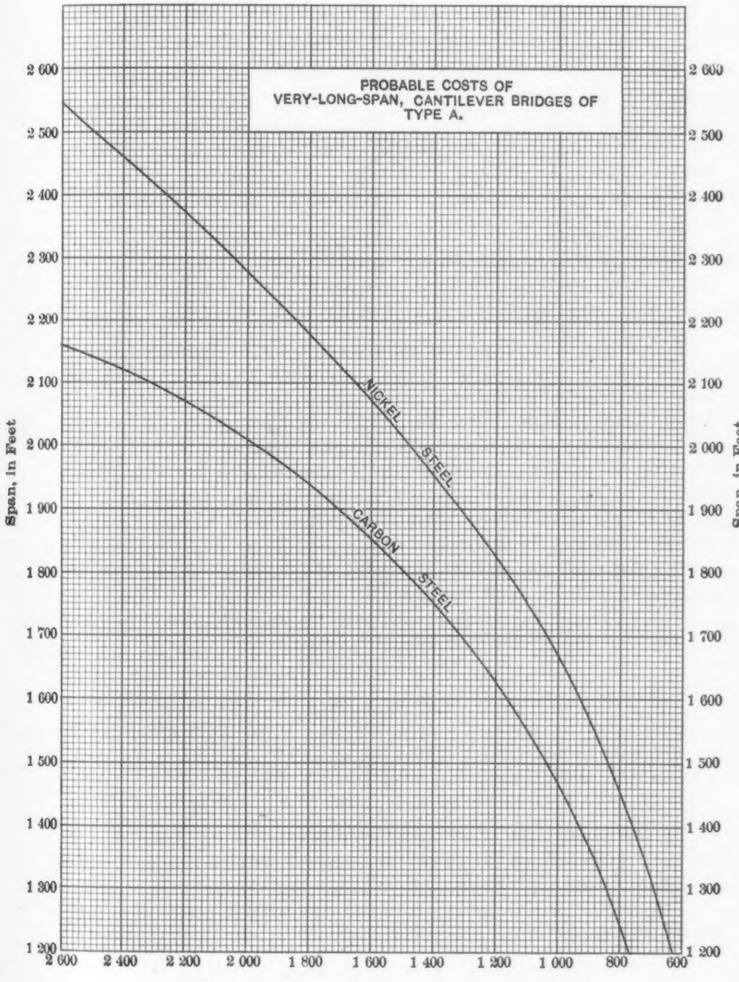

PROBABLE COSTS OF
VERY-LONG-SPAN, CANTILEVER BRIDGES OF
TYPE A.
NICKEL STEEL
CARBON STEEL
Span, in Feet
2 600
2 500
2 400
2 300
2 200
2 100
2 000
1 900
1 800
1 700
1 600
1 500
1 400
1 300
1 200
2 600
2 400
2 200
2 000
1 800
1 600
1 400
1 200
1 000
800
600
Cost of Metal Erected, in Dollars per Linear Foot of Span.

FIG. 72.

APPENDIX A. PART I.

COMPARATIVE TESTS OF STRUCTURAL NICKEL STEEL AND MEDIUM CARBON STEEL.

Preliminary Work.—In November, 1903, this investigation had its inception, and, early in December, two parallel series of tests were made at the American Bridge Company's plant at Pencoyd, Pa. For this preliminary work, two steels of the following composition were used:

	Heat No. 16 080	Heat No. 17 065.
Nickel	3.21	4.25
Carbon	0.390	0.463
Manganese	0.65	0.67
Sulphur	0.015	0.014
Phosphorus	0.011	0.019

A full report of this examination was made in January, 1904, and the results were so promising that immediate steps were taken for a more elaborate series of tests upon a steel made especially for this purpose. Arrangements were made with the Carnegie Steel Company, in the same month, for two melts of nickel steel.

After many delays it was finally decided to have the "shape" nickel steel rolled and tested before ordering the "eye-bar" nickel steel. Two melts of almost identical composition were made at the Homestead Works, Carnegie Steel Company; the first, Heat No. 17 673, on November 1st, and the second, Heat No. 17 749, on December 1st, 1904, each being of basic open-hearth steel.

The following material was shipped to Pencoyd, Pa., for testing during December, 1904, and January, 1905:

1 Universal plate,	12 by $\frac{3}{8}$ in.,	length	17 ft.
1 " "	12 by $\frac{1}{2}$ in.,	"	17 ft. 7 in.
1 " "	12 by $\frac{3}{4}$ in.,	"	15 ft. 1 in.
1 " "	12 by 1 in.,	"	17 ft.
5 angles,	6 by 6 by $\frac{3}{4}$ in.,	"	15 ft.
1 "	8 by 8 by 1 in.,	"	15 ft.

At the same time other material was shipped to Ambridge, Pa., for the fabrication of struts for compression tests and of eye-bars for "full-sized" tests.

The medium-carbon steel plates for comparative tests were not received at Pencoyd until December, 1905. They were as follows:

1 Universal plate,	12 by $\frac{3}{8}$ in.,	length,	20 ft. $\frac{1}{2}$ in.
1 " "	12 by $\frac{1}{2}$ in.,	"	20 ft. 1 in.
1 " "	12 by $\frac{3}{4}$ in.,	"	19 ft. $11\frac{1}{2}$ in.
1 " "	12 by 1 in.,	"	19 ft. $8\frac{1}{2}$ in.

There were no tests of angles of carbon steel.

Composition of Steels.—Several analyses were made of both the nickel-steel and carbon-steel heats, as shown in Table 7.

TABLE 7.—ANALYSES OF NICKEL AND CARBON STEEL.

NICKEL STEEL. HEAT NO. 17 673.

	Nickel.	Carbon.	Manganese.	Sulphur.	Phosphorus.	Silicon.	Analyst.
Desired.	3.50	0.38	0.70	below 0.04	below 0.08	below 0.04	
1	3.66	0.37	Grav. 0.80	0.025	0.010	0.050	Mill.
2	3.64	0.37	" 0.80	0.023	0.010	0.047	"
3	3.66	0.36	" 0.76	0.024	0.010	0.050	"
4	3.68	0.408 0.407	Comb. 0.758	0.020	0.005 0.005	Grav. 0.046 Vol.	Shimer, Bethlehem.
5	3.87	0.471	0.771	0.019	0.010	0.039	Booth, Garrett and Blair.
6	3.92	0.417	....	0.021	0.006	0.035	Orford Works.

Nos. 1, 2, and 3 were ladle analyses, made at the mill during pouring, in the order named; No. 1 between the first and second ingots, No. 2 between the fourth and fifth ingots, No. 3 between the ninth and tenth ingots. The carbon was probably determined by gravimetric analyses.

No. 4 was made from drillings taken from test ingot No 2.

Nos. 5 and 6 were made from duplicate drillings taken from ¾-in. plate.

NICKEL STEEL. HEAT NO. 17 749.

	Nickel.	Carbon.	Manganese.	Sulphur.	Phosphorus.	Silicon.	Analyst.
1	3.50	0.36 color.	0.82	0.027	0.011	0.060	Mill.
2	3.50	0.39 comb.	0.78	0.033	0.015	0.060	"
3	3.50	0.38 color.	0.81	0.028	0.011	0.048	"
4	3.52	0.356 comb.	0.763	0.030	0.012	0.058	Shimer.

Nos. 1, 2, and 3 were ladle analyses, made at the mill during pouring, in the order named; No. 1 between the second and third ingots, No. 2 between the tenth and eleventh ingots, No. 3 between the twelfth and thirteenth ingots.

No. 4 was made from drillings taken from test ingot No. 2.

CARBON STEEL. HEAT NO. 33 342.

	Nickel.	Carbon.	Manganese.	Sulphur.	Phosphorus.	Silicon.	Analyst.
1	....	0.253	0.546	0.025	0.014	0.016	Booth, Garrett and Blair.
2	....	0.198	0.55	0.025	0.011	0.012	Orford Works.

Nos. 1 and 2 were made from duplicate drillings taken from ¾-in. plate.

CARBON STEEL. HEAT NO. 41 520.

	Nickel.	Carbon.	Manganese.	Sulphur.	Phosphorus.	Silicon.	Analyst.
1	....	0.19	0.54	0.033	0.022	....	Mill.
2	....	0.287	0.563	0.025	0.025	0.020	Booth, Garrett and Blair.
3	.. ..	0.234	0.62	0.024	0.023	0.035	Orford Works.

Nos. 2 and 3 were made from duplicate drillings taken from ¾-in. plate.

For use in the fabrication of struts and for the 12-in. universal plates, tested for comparison with the nickel steel, a medium carbon steel of good quality was required.

All the 12-in. plates for specimen tests were rolled from Nickel-Steel Heat No. 17 673 and Carbon-Steel Heat No. 33 342. The angles for specimen tests and the greater part of the material for the struts were rolled from Nickel-Steel Heat No. 17 749 and Carbon-Steel Heat No. 41 520.

A comparison of the analyses of the steels shows that the material obtained was as close to the desired composition as it was practicable for the mill to make. The tests made on the plates did not reveal any lack of homogeneity.

The surface of the nickel-steel plates was smooth and free from scale, much more so than that of the carbon-steel plates. The surface of the nickel-steel angles was not so good, as in one or two instances the fin from the edge and some scale had been rolled in.

All the material shipped to Pencoyd and Ambridge was stored unprotected from the weather for several months, the best means for identification later, being the cleaner and smoother surface of the nickel steel.

Tests of 12-in. Universal Rolled Plates and Angles.

Tensile Tests—Plain Specimens.—The test given the greatest attention in mill practice everywhere is the tensile test. In the absence of inspection, or when, for any reason, knowledge of the physical properties of material is desired by the mill, a tensile test only is made. At many mills no other test is made by the inspector for his client. For these reasons, a large number of specimens were prepared from this structural nickel steel and from the structural carbon steel tested for comparative purposes.

Because of the uncertainty attached to the usual method of determining the "yield point" or so-called "elastic limit," arrangements were made for two series of tests, one on parallel-sided pieces and one on pieces having edges machined parallel for a distance of 9½ in., only, with fillet widening pieces at the ends, and known as the standard adopted by the American Association of Steel Manufacturers (abbreviated here into A. A. S. M.). The length of all specimens was 18 in., and the width, except for those cut from the 1-in. material, was 1½ in. The width of these was reduced to 1¼ in. because it was expected that a 150 000-lb. machine would be used to make the tests, and it was necessary to keep the ultimate strength well within this limit.

The "parallel-sided" pieces were to be tested according to mill practice at a representative mill. At the suggestion of Mr. C. L. Huston, of the Lukens Iron and Steel Company, Coatesville, Pa., half these pieces were tested at this mill, and the others at the Pencoyd Iron Works,

Pencoyd, Pa. Mill practice was not followed exactly, because it was not thought possible to get even a close approximation for the "yield point" of the nickel steel, with the high speeds in every-day use. The only change, however, was in the reduction of the speed of breaking.

The A. A. S. M. pieces were to be tested in a machine equipped with an autographic attachment; and a 200 000-lb. Olsen machine, at Drexel Institute, Philadelphia, was used, through the kindness of Mr. Earl B. Smith, Instructor in Mechanical Engineering. All the machine work was done at the Pencoyd Iron Works.

The tensile tests in the original lay-out were located at the ends of the plates and angles, the A. A. S. M. pieces at the edge, and the parallel-sided pieces inside and adjacent. As it was desirable to repeat certain tests to bring out more definitely points of difference between the two steels, additional pieces had to be selected from available material. The tensile strength of plates varies with the location of the specimen in the plate and of the slab in the ingot, so this forced selection of additional pieces added necessary information on these points. All the material was rolled from slabs; hence these variations in physical properties due to location are not so pronounced as in material rolled directly from the ingot.

American Association of Steel Manufacturers' Specimens.—The results of these tests are given in detail in Tables 30 and 31, and a comparison with the results of tests on parallel-sided specimens may be found in that part of this appendix filed for reference in the Library of the Society.

The speed of machine or movement of head for the first eleven tests was 1 in. in 6 min., but, while this gave good results for the carbon steel, it was too rapid for the nickel steel, and the yield point was lost. The weight could not be made to travel along the beam with sufficient rapidity to keep the beam balanced, and, consequently, the pencil recorded a faulty curve. A speed of 1 in. in 6 min. was desired in order to economize time, the speed, 1 in. in 20 min., adopted for the remaining pieces, requiring ½ hour for each test. In these remaining tests, up to a point just below the yield point, a speed of 1 in. in 6 min. was used, and then a speed of 1 in. in 20 min. until the material had well passed this point, when 1 in. in 6 min. was again used. In several tests, when the maximum strength had been developed, a still faster speed (1 in. in 2 min.) was used for breaking the specimen. The speed at breaking was found to affect the tensile strength materially.

Yield Point.—The term "yield point" has been adopted herein in place of "elastic limit" because it more nearly represents what was sought, namely, the point at which the material would no longer return to its original condition upon the removal of the load. It is shown graphically in the autographic diagram by the departure of the record from a straight line.

The determination of the true elastic limit was not possible with the apparatus at hand.

The loss of time caused by the use of an autographic attachment bars the adoption of this method in mill practice, yet it was desired that the yield point reported in these tests should be a point easily obtainable in practice by a skillful operator, and not one depending upon time-consuming and expensive devices for its exact location. It was assumed to be the "least permanent set" visible in a specimen by the aid of a pair of finely pointed dividers. The point in the curves corresponding to this "least permanent set" was obtained after some experimenting, as shown in Table 8.

TABLE 8.—BEAM READINGS.

Mark.	Drop of beam, in pounds.	Yield point, in pounds.	Load released at:	Permanent set in 6 in. by dividers:
CTSL 150	38 000	35 200	36 600 lb.	0.01 in.
" 193	36 600	34 800	36 000 "	0.02 "
" 198	39 400	33 800	33 200 "	0.01 "
" 152	34 200	33 000	33 600 "	0.015 "
TSL 3	....	38 600	38 000 "	0.005 "
" 40	....	39 200	39 000 "	0.015 "
" 43	....	40 600	40 000 "	0.01 "
" 44	....	38 800	39 000 "	0.015 "
" 52	....	46 500	48 000 "	0.02 "
" 70	....	47 100	46 900 "	0.005 "
" 89	....	47 500	48 000 "	0.02 "
" 109	....	70 500	69 300 "	0.00 "
			75 000 "	Between these readings a rapid elongation of specimen took place.
" 150	....	77 400	77 800 "	0.025 in.
" 154	....	76 600	77 400 "	Appreciable.
" 199	....	73 000	72 000 "	Not appreciable.
			76 000 "	0.01 in.
ATSL 58	....	69 200	64 400 "	Not appreciable.
	....	....	70 600 "	Not appreciable.
	....	....	75 000 "	0.02 in.

In the tests given in Table 8 the load was removed at about the yield point, the permanent set, if any, was measured, and the movement of the pencil noted. If this traced a new line parallel to the original curve, there was visible evidence of a permanent set and of the amount. To check this, careful measurements were made by dividers. It was assumed that, for all practical purposes, a set of 0.01 in. in 8 in. was the "least visible," and the point in the curve corresponding to this was obtained graphically at the intersection of a line 0.01 in. away from, and parallel to, the straight portion of the record. The stretch shown by the curve is the stretch in 8 in. It was found that, even when the record had departed from a straight line, if approximately this distance of 0.01 in. had not been exceeded, on releasing the load the pencil would retrace the original line, and no elongation could be detected by the dividers.

At times the mechanism of the recorder did not work properly, but usually these vagaries were easily corrected. The movements of the weight and of the pencil were constantly checked, one with the other. The yield point thus determined is slightly lower than the drop of beam. In tests of nickel steel, of this content of nickel and carbon, the beam does not drop at the point where the stretch continues without an increasing load, because there is no such point. The autographic curves show this; for it will be seen that in no curve of nickel steel is there a bend at the yield point of even approximately 90 degrees. A "drop" will occur if the weight is run out uniformly, for the rate of stretch changes, but it will be almost invariably at too great a load as registered by the weight on the beam, for in mill practice the speed is so great that the beam is never balanced until after the so-called elastic limit or yield point is reached. In carbon steel an opportunity is given, by the stretching of the steel, for the weight to "catch up"; in nickel steel there is none. This subject will be mentioned later.

The elongations in 2, 4 and 6 in. are given; although their value is not great, they show that considerable stretching takes place over the entire length of the specimen.

The "area of fractured section" was measured at points midway between edges at Pencoyd and Drexel. This method is followed in most mills, but it gives slightly higher percentages of reduction than actually exist. At Lukens, edge measurements were taken, that being the practice at that mill. The dimensions of this section, and also of the original section, are given to the nearest 0.005 in., this being near enough for all practical purposes. At Pencoyd and Drexel the elongations were measured along the edge of the specimen; at Lukens, along the middle. This difference affects slightly the comparison of results. The elastic ratio is the ratio of yield point or drop of beam to the ultimate strength.

In Table 31 both drop of beam and yield point, for specimens broken at Drexel Institute, are given. The "stretch at the drop of beam," in Table 31, has no special significance, except to show why mill methods will give fair results for testing carbon steel and not for nickel steel, in which no such stretch occurs. It was measured from the autographic records. The drop of beam was taken from the cards as that load at which the curve first became horizontal. Actually, no drop of beam occurred, as the machine was equipped with a device for controlling automatically the movement of the weight; and, when adjusted properly at the slow speed used, the beam was balancing throughout.

Parallel-Sided Specimens.—The tests on parallel-sided specimens are interesting, as showing how close an approximation to the true yield point may be obtained by the drop of the beam, by expert operators of mill-testing machines.

A 200 000-lb. Olsen machine was used at each mill. At Pencoyd the weight was run out on the beam by an electric motor, excited by contacts at the end of the beam, and, at Lukens, by a hand-wheel. Operating by hand is shown to be much more sensitive, the reason being easily seen by one witnessing a test. When testing the nickel steel, the Pencoyd operator had to know in advance about where to expect the drop. Even with this information, the results are largely guesswork. The Lukens operator did not need this information, and, while the beam readings are a little high, for the reasons explained, the results are in fair agreement with those obtained with the autographic recorder, and the time for making the tests is brought within the possibilities of mill practice. The speeds used at each mill for nickel steel were reduced below those for carbon steels.

Examination of Results—Nickel Steel—Tables 30 and 32.

A fair degree of uniformity exists throughout.

Yield Point.—Edge specimens have a higher yield point than corresponding interior pieces. Specimen tests made at Lukens agree closely with those made at Drexel, but those made at Pencoyd are generally higher. The yield point becomes lower as the thickness of the material increases.

Material Speed, 1 in. in:	⅜-in. plate. 20 min.	½-in. plate. 20 min.	¾-in. plate. 20 min.	1-in. plate. 20 min.	¾-in. angle. 20 min.	1-in. angle. 20 min.
Edge	70 100	......	62 100	58 600	62 300	54 200
Interior	67 100	62 100	......	58 500	......	......

Tensile Strength.—Edge specimens have a tensile strength about 3 000 lb. higher than corresponding interior pieces. All specimens cut from the same thickness agree closely when variations of speed and location are considered. The tensile strength becomes less as the thickness increases, also as the speed of breaking decreases.

TABLE 9.

Material...	⅜-in. Plate			½-in. Plate.			¾-in. Plate.		
Speed, 1 in. in:	6 min.	3-2 min.	40-10 sec.	6 min.	3-2 min.	40-10 sec.	6 min.	3-2 min.	40-10 sec.
Edge	116 600	116 300	......	110 600	113 800	......	106 800	108 400	......
Interior	113 900	112 600	116 400	......	107 500	112 500	......	108 100	111 200

Material....	1-in. Plate.			¾-in. Angle.			1-in. Angle.		
Edge	......	107 300	......	103 000	......	......	......	97 900	......
Interior	104 600	103 500	105 700	......	......	104 400	......	......	100 300

1-in. and ¾-in. plates were rolled from slabs 14½ by 9¼ in.
½-in. " ⅜-in. " " " " " 14½ by 8¼ in.
8 by 8 by 1-in. angles " " " " 11½ by 8 in.
6 by 6 by ¾-in. " " " " " 8 by 9 in.

The slabs were rolled from ingots 25 by 30 in.

The relative amounts of work done on the slab in rolling are shown in Table 10.

TABLE 10.—RELATIVE AMOUNTS OF WORK DONE IN ROLLING.

12-in. plates, thickness.	Area of slab.	Area of shape.	Percentage of reduction.
1 in.	134 sq. in.	12 sq. in.	91.0
¾ "	134 "	9 "	93.3
½ "	120 "	6 "	95.0
⅜ "	120 "	4½ "	96.3
Angles.			
8 × 8 × 1 in.	92 "	15½ "	83.1
6 × 6 × ¾ in.	72 "	8½ "	88.2

The differences in tensile strength due to thickness alone result probably from a combination of two varying conditions: first, the chilling during rolling; and, second, the amount of reduction of the slab. Differences in other physical properties necessarily arise from these same conditions.

Elastic Ratio.—The elastic ratio is seen to decrease uniformly as the thickness increases, varying from 60.1 for the ⅜-in. material to 54.5 for the 1-in. material.

Elongation and Reduction of Area.—The elongation and reduction of area increase slightly as the thickness of the material increases, but the variation lacks uniformity.

TABLE 11.—ELONGATION AND REDUCTION OF AREA.

MATERIAL........		⅜-IN. PLATE.			½-IN. PLATE.			¾-IN. PLATE.		
Speed, 1 in. in:		6 min.	3–2 min.	40–10 sec.	6 min.	3–2 min.	40–10 sec.	6 min.	3–2 min.	40–10 sec.
Elongation:	Edge....	....	15.7	....	16.5	16.4	....	17.5	18.1	......
	Interior.	15.1	16.5	15.6	....	17.1	16.5	......	16.5	17.8
Reduction of Area:	Edge....	....	46.4	....	46.9	46.0	....	48.1	47.4	......
	Interior.	45.6	48.9	45.9	....	50.0	47.3	......	43.6	40.7

MATERIAL........		1-IN. PLATE			¾-IN. ANGLE.			1-IN. ANGLE.		
Elongation:	Edge....	....	20.0	....	18.7	....	....	......	21.3	......
	Interior.	19.8	20.5	21.2	....	....	17.9	......	......	18.4
Reduction of Area:	Edge....	....	47.4	....	42.0	....	....	......	48.5	......
	Interior.	49.2	49.8	48.5	....	....	35.2	......	......	41.0

Speed of Machine.—The tensile strength is raised by an increase in speed; the effect on elongation and reduction of area of fracture is not marked; the tendency, however, is to reduce the reduction of area.

EXAMINATION OF RESULTS—CARBON STEEL—TABLES 31 AND 33.

From only one thickness of material, 1-in. plate, were specimens cut from both edge and interior, hence remarks as to the effect of location are limited to this one example.

Yield Point.—Edge specimens have a yield point slightly higher than the corresponding interior pieces.

Drop of Beam.—

The average drop of beam for all Drexel tests is 34 100 lb.
" " " " " " " Lukens " " 32 800 "
" " " " " " " Pencoyd " " 40 600 "

The speed of machine differed in each case, being 1 in. in 6 min. at Drexel, 1 in. in 3 min. at Lukens, and 1 in. in 15 sec. at Pencoyd, yet the close agreement between the two former and the great difference of the Pencoyd readings show how little dependence can be put upon results obtained with the machine running at a high speed.

The average of all the Drexel tests for the yield point is 32 800, which is 1 300 lb. lower than the corresponding drop of beam.

There is a close uniformity between the results obtained at the same place.

	⅜ in. plate.	½-in. plate.	¾-in. plate.	1-in. plate.		
Yield point at Drexel.....	38 600	36 900	33 400	27 800	(26 100)*	Edge.
Drop of beam at Drexel..	40 800	37 300	34 100	30 300	(27 000)*	Edge.
Drop of beam at Lukens.	38 000	33 900	30 900	28 500	....	Interior.
Drop of beam at Pencoyd	46 700	40 800	36 400	38 500	....	Interior.

* Interior specimens.

These figures show how regular is the decrease in the yield point as the thickness of the material increases. The same conditions probably exist as influenced the physical properties of the nickel steel. The edge specimens cut from the 1-in. plate have a higher yield point than the corresponding interior ones.

Tensile Strength.—Edge specimens have a tensile strength about 3 000 lb. higher than the corresponding interior pieces.

MATERIAL...	⅜-IN. PLATE.			½-IN. PLATE.			¾-IN. PLATE.			1-IN. PLATE.		
Speed, 1 in. in:.	6 min.	3–2 min.	15–10 sec.	6 min.	3 min.	15–10 sec.	6 min.	3 min.	15–10 sec.	6 min.	3–2 min.	15–10 sec.
Edge	......	63 700	......	63 800	......	......	62 200	......	......	......	61 900	......
Interior	......	62 400	66 100	......	62 000	65 300	......	61 100	62 700	59 000	58 300	61 600

The differences due to thickness are evident, but they are small, and those due to speed of breaking and of location in plate are well marked.

Elastic Ratio.—The elastic ratio decreases as the thickness of material increases, varying from 60.8 for the ⅜-in. material to 45.0 for the 1-in. material.

Elongation and Reduction of Area.—The elongation seems to increase slightly as the thickness increases, but the effect is small, and it is not noticeable at all on the reduction of area. An increase in the speed of breaking causes a decrease in the reduction of area, although this is not well marked, and the effect on the elongation is not noticeable.

Material		⅜-in. Plate.			½-in. Plate.			¾-in. Plate.			1-in. Plate.		
Speed, 1 in. in:		6 min.	3–2 min.	15–10 sec.	6 min.	3 min.	15–10 sec.	6 min.	3 min.	15–10 sec.	6 min.	3–2 min.	15–10 sec.
Elongation	Edge	....	28.0	....	27.4	....	....	22.2	....	....	....	31.9	....
	Interior	....	28.5	30.3	....	30.5	28.0	....	32.5	32.7	33.5	31.0	34.4
Reduction of area	Edge	....	56.2	....	56.5	....	....	58.0	....	....	....	56.7	....
	Interior	....	59.5	56.1	....	59.2	55.3	....	59.1	55.9	60.4	61.4	57.2

Comparison of Nickel Steel and Carbon Steel.

The varying conditions affecting the physical properties of steel seem to have no greater effect on one metal than on the other, except upon the tensile strength. The greater relative reduction in strength of nickel steel with increasing thickness of material causes the elastic ratio for the thicker metal to be higher for the nickel steel than for the carbon steel, as shown by the comparisons in Table 12.

TABLE 12.—Elastic Ratio.

	⅜-in. plate.	½-in. plate.	¾-in. plate.	1-in. plate.	Tested at:	Speed, 1 in. in:	Location.
Nickel steel	60.1	58.0	57.8	*54.5	Drexel.	6 min.	Edge.
Carbon steel	*60.8	57.8	53.7	*45.0	"	6 min.	Edge.
Nickel steel	60.0	56.3	62.6	57.0	Lukens	3 min.	Interior.
" "	58.1	55.9	56.1	.....	"	10 sec.	"
Carbon steel	*64.4	58.5	54.8	*49.0	Drexel.	6 min.	Edge.
" "	60.7	55.2	51.1	48.7	Lukens	3 min.	Interior.
" "	58.6	51.4	48.7	46.1	"	10 sec.	"

* Speed, 1 in. in 2 min.

Nickel steel is not as ductile as carbon steel, yet it is not brittle in any sense. The yield point, or, as it is usually called, the elastic limit, may be safely taken as a minimum at 60 000 lb., as compared with the 30 000 lb. usually specified for carbon steel. In tests of the 1-in. material, a lower value was obtained, but the speed was slower

than in mill practice; and the carbon steel, tested similarly, gave also a lower value than 30 000 lb.

The ultimate strength varies with the thickness, speed, and the location of the specimen, but, as compared with the results of carbon steel, a value between 100 000 and 120 000 lb. may be obtained, and even 110 000 lb. as a minimum, if the conditions mentioned are not restricted—and it is not usual to restrict them.

An elongation of 15% in 8 in., and a reduction of area of 40% may be safely taken as the minimum. In almost every instance the fractures were silky, uniform, and free from laminations, pipe, etc. The only variation was in four of the plate tests and in the $\frac{3}{4}$-in. angle, these fractures being partly fine crystalline or granular.

The usual high elastic ratio for nickel steel is not found in these tests, probably because the speed of machine was much slower than is generally used, and the determination of the yield point was more accurate. As already stated, the methods in vogue at most mills would give a very high but inaccurate value for the yield point or elastic limit of nickel steel.

The tests were made at Drexel in February and March, at Pencoyd in March, and at Lukens in April, 1906.

Tensile Tests of Punched, Reamed, and Punched-Riveted Specimens.

These tensile tests were arranged, as far as possible, to show relatively what changes are effected in the physical properties of nickel steel and carbon steel by the shop operations of punching holes full size, of sub-punching and reaming to full size, and of riveting punched work. The full-sized holes were $\frac{15}{16}$ in. in diameter, the sub-punched holes, $\frac{11}{16}$ in. in diameter; $\frac{1}{8}$ in. of material, therefore, was reamed away.

It is hardly necessary to repeat here that the usually accepted opinions are that shearing the metal, as in punching, forms numerous incipient cracks radiating from the edge, and also that the material around the hole is hardened by the pressure necessary to force the punching out. It is believed that if these cracks and this hardened ring are cut away, all injury done to the material is removed. The opinions as to the effect of riveting are, perhaps, not so positive, because the subject has not been investigated experimentally; it was only introduced into this series after other tests had been made. The object was to determine the annealing effect, if any, on the material due to the driving of the hot rivet.

Test specimens, 3 in. wide and 18 in. long, were cut from the $\frac{3}{8}$-in. and the $\frac{3}{4}$-in. plates. The "punched" and the "reamed" pieces were laid out side by side in every instance, in order to give results as directly comparable as possible; the "punched-riveted" pieces were cut from available surplus material. Through an error, one piece of

carbon steel, which it was intended to punch full size, was sub-punched and reamed, and *vice versa*. The edges of all holes and plates were rounded slightly by filing to remove burrs.

Pieces *TPPL* 13 and 33, *TAPPL* 46, 140, and 142 were cut from the edge of the plate, but this does not seem to have influenced the results as in the tensile tests of plain specimens, probably because of the greater width of 3 in. The punching, reaming, and riveting were done in the bridge shop at Pencoyd in the customary way. The diameter of the die used in the punching was $\frac{1}{16}$ in. larger than that of the punch. To prevent injury to the material in cutting off the heads of rivets, the latter were driven with a plate, 3 by $\frac{1}{16}$ by 9 in., on each side of the specimen. These plates served another purpose, not intended, namely, the lessening of the amount of compression in the material under the head of the rivet, caused by the pressure of the cup of the riveting machine. The pressure on the rivet in driving was probably about 30 tons per sq. in., and the annealing effect, if any, was entirely destroyed by the compression around the hole. The metal under the head was bright, and the calipering showed a reduction of thickness averaging as follows:

	Carbon steel.	Nickel steel.
$\frac{3}{8}$	0.003 in.	0.003 in.
$\frac{3}{4}$	0.007[illegible] "	0.003 "

All the testing was done on the testing machine at Drexel Institute, and the autographic recorder was used to plot the curve of each test; and from this record the elastic limit, yield point, and ultimate strength were determined.

Yield Point.—As in the tensile tests of plain specimens, at about the yield point, the load was removed from several specimens and the permanent set carefully measured by dividers and checked with the card record, the results shown in Table 13 being obtained:

TABLE 13.—BEAM READINGS.

Mark.	Yield point, in pounds.	Load released at: Pounds.	Permanent set in 6 in., by dividers: Inches.
CTAPPL 46	29 500	21 000	None.
		30 200	Widened line.
" 140	66 500	66 000	None.
		68 000	Widened line.
		72 000	0.05
" 142	68 000	70 000	0.02
TAPPL 46	62 000	46 700	None.
		63 000	None.
		73 000	0.03
" 140	128 000	112 000	Widened line.
" 142	120 000	120 000	0.01
Heat No. 16 080:			
TPPL 3/8	53 200	54 000	0.01 scant.
TPPL 3/4	100 000	96 000	Widened line.
		104 600	0.02 scant.
TPRL 3/8	52 000	52 000	Widened line.
		56 000	0.02 scant.
TPRL 3/4	98 800	90 000	Widened line.
		102 800	0.015

The yield point, as before, was obtained from the curve at the intersection of a line 0.01 in. away from, and parallel to, the straight portion of the record. While there is a fair agreement between the results obtained by the two methods described, it is not as close as in the tensile tests on plain specimens, and an examination of the curves will show the reason. The deflection from a straight line is so gradual and, in those of the nickel steel, so slight, that it is impossible to obtain, by any practical means, the exact point where a permanent set first takes place. Yet the loads recorded in these tables are closely approximate (with two or three exceptions mentioned later), so that, for comparative purposes, the data obtained are of sufficient value.

In all the curves of the carbon steel there is a decided change of direction, corresponding to the stretch occurring at the yield point for the plain specimens. In these tests this change varies in form and position. In the tests of the $\frac{3}{8}$-in. material it occurs just before the maximum strength is developed; and, in those of the $\frac{3}{4}$-in. material, it is found close to the yield point. There is a sufficiently marked change in the curve at what has been taken as the yield point to substantiate the tabulated figures. In the curves for tests from Heat No. 16 080, similar changes occur, but not in the other nickel-steel tests —the former material is a softer grade of steel. The $\frac{3}{4}$-in. material is for all steels softer than the $\frac{3}{8}$-in. material; the variations described may be assumed, therefore, as being conditional upon the grade of steel.

All these tests were made in a similar manner, a speed of 1 in. in 20 min. being used just before and until the yield point was well passed, and a speed of 1 in. in 6 min. at the beginning and at the instant of breaking.

Tables 34, 35, and 36 show the results of each test in detail.

1. Curve *CTPRL* No. 7 differs from the other three in that it has a higher yield point and is straighter within this point. The autographic attachment did not work properly.
2. Curve *TPRL* No. 7 is slightly straighter within the yield point than the other three curves, but it is not abnormal.
3. Curve *TPPL* No. 111 differs for the same reason—the curve is flatter.
4. Curves for *CTAPPL* are all irregular; the autographic attachment did not work properly in describing any of these curves, but the yield point is well marked, so that the results are fairly accurate.
5. Curve *TAPPL* No. 141 differs from the others for the same reason. These curves are all so flat that it is practically impossible to obtain the yield point accurately.

The other curves are all normal.

Examination of Results.—Tables 34, 35, and 36.

In the comparisons in Tables 14 and 15 it must be borne in mind that these tests were made on specimens 3 in. wide, while the plain specimens were but $1\frac{1}{2}$ in. wide. It is well known, also, that short grooved specimens give a higher ultimate strength than the usual 18-in. specimen. It is not probable, therefore, that all the effects described are the direct result of alterations in the structure of the material about the hole.

TABLE 14.—PERCENTAGES BASED ON RESULTS OF TESTS OF REAMED SPECIMENS.

Steel.	Physical property.	⅜-in. Material.			¾-in. Material.		
		Reamed.	Punched.	Punched-riveted.	Reamed.	Punched.	Punched-riveted.
Nickel..	Yield point............	95	102	113	114	111	130
Carbon.	" "	102	105	107	112	124	128
Nickel..	Ultimate strength.....	101	91	94	104	89	82
Carbon.	" "	103	93	93	107	94	81
Nickel..	Elastic ratio..........	94	112	121	110	125	157
Carbon.	" "	100	113	115	104	132	158

TABLE 15.—PERCENTAGES BASED ON RESULTS OF TESTS OF PLAIN SPECIMENS.

Steel.	Physical property.	⅜-in. Material.		¾-in. Material.	
		Punched.	Punched-riveted.	Punched.	Punched-riveted.
Nickel	Yield point..........................	108	120	97	113
Carbon	" "	103	105	111	114
Nickel	Ultimate strength	91	93	85	79
Carbon	" "	91	91	88	75
Nickel.......	Elastic ratio	120	130	114	143
Carbon	" "	114	115	126	152
Nickel	Elongation in 4 in..................	62	91	50	46
Carbon	" "	77	80	39	14
Nickel.......	Reduction of area..................	51	57	39	22
Carbon	" " "	75	64	28	3

Punching, sub-punching-and-reaming, and riveting, in every instance, alter the $\frac{3}{4}$-in. material more than they do the $\frac{3}{8}$-in. material, in both nickel and carbon steels.

Punching $\frac{15}{16}$-Inch Hole.—Punching raises the yield point and lowers the ultimate strength. The yield point of the nickel steel is affected to a smaller extent than that of the carbon steel, and the ultimate strength more, but the difference is not great.

Reaming—From $\frac{11}{16}$-Inch to $\frac{15}{16}$-Inch.—The process of sub-punching-and-reaming raises both the yield point (with but one exception,

namely, that of the ⅜-in. nickel steel) and the ultimate strength of both steels, and in almost the same proportion. The effect is less than that of punching full size, probably because the punched hole is smaller, and the injury, therefore, less, and because the reaming removes the greater part of the affected area. Nickel steel, on the whole, is affected to a smaller extent than carbon steel, but the difference is not uniform, and is small. The elongation and reduction of area cannot be compared directly with those of the plain specimens because of the marked difference in the shape and character of the specimen. Comparing the results of punching with those of sub-punching-and-reaming, however, shows that the ductility of the material is injuriously affected by punching without subsequent reaming. The difference between the effects of punching and reaming upon the ⅜-in. nickel steel is greater than it is upon the corresponding carbon steel, but it is less upon the ¾-in. material.

Riveting.—Riveting raises still further the yield point, and lowers the ultimate strength. The elongation and reduction of area are also smaller than for the reamed specimens. Upon the ⅜-in. material, however, riveting seems to be beneficial, but upon the ¾-in. material it adds to the injury resulting from punching. Nickel steel is affected much less by riveting than carbon steel, probably because its resistance to compression is greater.

The ¾-in. material being a softer steel than the ⅜-in., it was to be expected that the effect of riveting upon it would be greater.

In comparison with the carbon steel, the nickel steel in these special tests shows up very favorably, on the whole.

TABLE 16.

Kind of specimen.	⅜-in. Material. Area = 1.125 sq. in.			¾-in. Material. Area = 2.25 sq. in.		
	Actual yield point.	Actual ultimate strength.	Elastic ratio.	Actual yield point.	Actual ultimate strength.	Elastic ratio.
Nickel Steel.						
Plain specimens	76 000	128 000	59	140 000	243 000	58
Reamed "	51 000	91 000	56	110 000	174 000	63
Punched "	55 000	82 000	67	106 000	147 000	72
Punched-riveted specimens.	61 000	85 000	72	124 000	137 000	90
Carbon Steel.						
Plain specimens	42 000	70 000	60	72 000	137 000	53
Reamed "	29 000	48 000	60	57 000	102 000	56
Punched "	30 000	43 000	70	64 000	90 000	71
Punched-riveted specimens.	30 000	43 000	70	67 000	78 000	86

The comparisons (in Table 16) of the actual yield point and ultimate strength of these specimens with those of the plain specimens, on the basis of a sectional area of 3 by ⅜-in. and 3 by ¾-in. show that the loss of strength is less for the nickel steel in ⅜-in. material than for the corresponding carbon steel, and greater for the ¾-in. material. The differences are small.

TABLE 17.—PERCENTAGE OF LOSS, BASED ON PLAIN SPECIMENS.

Steel.	Physical Property.	⅜-in. Material. Reamed.	⅜-in. Material. Punched.	⅜-in. Material. Punched-riveted.	¾-in. Material. Reamed.	¾-in. Material. Punched.	¾-in. Material. Punched-riveted.
Nickel	Yield point	33	28	20	21	24	11
Carbon	"	31	29	29	21	11	7
Nickel	Ultimate strength	29	36	34	28	40	44
Carbon	"	31	39	39	26	34	43

TABLE 18.—COMPARISON OF ASSUMED YIELD POINT WITH ELASTIC LIMIT.

(In Pounds per Square Inch.)

Material.	Physical property.	Nickel Steel. Plain.	Nickel Steel. Reamed.	Nickel Steel. Punched.	Nickel Steel. Punched-riveted.	Carbon Steel. Plain.	Carbon Steel. Reamed.	Carbon Steel. Punched.	Carbon Steel. Punched-riveted.
⅜-in	Elastic limit	60 000	49 000	50 000	55 000	35 000	28 000	30 000	27 000
	Yield point	67 000	64 000	69 000	77 000	39 000	40 000	41 000	41 000
¾-in	Elastic limit	53 000	53 000	51 000	61 000c	29 000	26 000	32 000a	35 000b
	Yield point	62 000	71 000	69 000	80 000	33 000	37 000	42 000	43 000

a. No. 126 not included.
b. Pencil of autographic recorder did not work properly; the figures, therefore, may not be accurate,
c. Curves are so flat that the figure may not be accurate.

Table 18 shows that within the yield point there are changes in the material, as shown in the autographic curves. The departure of the record from a straight line is what is meant by elastic limit.

The number of tests was too small, and the apparatus used was not sufficiently delicate to furnish much more than a suggestion that these operations, necessary to the fabrication of bridge members, weaken the perfect elasticity of the material. The ductility is also lessened. The method used for determining the yield point in these tests does not give such uniform or accurate results as in the tests of plain specimens. In the specimens, the ratio of stretch to the load increases very

gradually in these tests, especially in the case of nickel steel, the records of which are almost straight.

The exceptional results shown for the ¾-in. material may have been caused by the bending of the test pieces during punching and riveting. This treatment would have a tendency to raise the recorded value for the elastic limit, though actually it would be less. The results for the punched-riveted specimens of ¾-in. material may be inaccurate as noted.

The fractures are interesting. The ⅜-in. material, on the whole, was silky, though the nickel steel in the punched and riveted specimens was partly crystalline.

The ¾-in. material of nickel steel was fine crystalline, as was also that of carbon steel, with the exception of the reamed specimens. The nickel steel, however, was much finer-grained, and showed more plainly lines like magnetic lines radiating from the hole. Under the skin the fracture was silky, and this was more marked in the nickel steel. The material near the hole was also silky, but this was more marked in the carbon steel.

The break occurred, with two or three exceptions, simultaneously on both sides. The difference in any case was only a fraction of a second. Those with a crystalline fracture were always unexpected, with no drawing down of the breaking section.

The tests of the punched and of the reamed specimens were made in March, and those of the riveted specimens in June, 1906.

Bending Tests, Plain Specimens.

Next to the tensile test, the test most frequently made is the bending of a specimen piece of steel until it is proved that the ductility of the material is sufficient to satisfy the specifications. This test is also an indication of the absence of high phosphorus.

The bending may be done in various ways: On a U-shaped block, under a steam hammer, by being pushed through an opening in an anvil by a plunger, or, in the case of iron and soft steel, by being wrapped around a mandrel, one end of the specimen being held rigidly and the power being applied at the other end by an eccentric cam.

This last method makes the most perfect bend, but for heavy sections of medium-carbon steel and especially of nickel steel, it is not practicable. The method to be followed is not often specified, yet it is an important factor, as is the speed of machine in the tensile test or the taper of the drift-pin in the drifting test.

These tests were made in the testing room of the Lukens Iron and Steel Company through the kindness of Mr. C. L. Huston, Vice-President, and Mr. Howard Taggart, Engineer of Tests.

The specimen was laid on an anvil over a rectangular opening, wider at the top than at the bottom. The piece was bent by being

PLATE XV.
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NICKEL STEEL FOR BRIDGES.

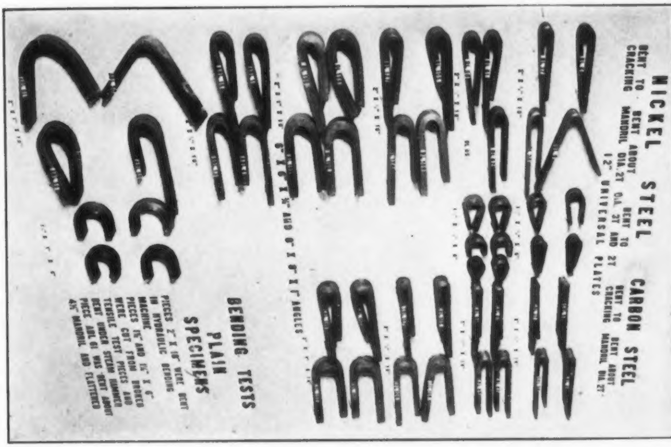


FIG. 1.—BENDING TESTS.

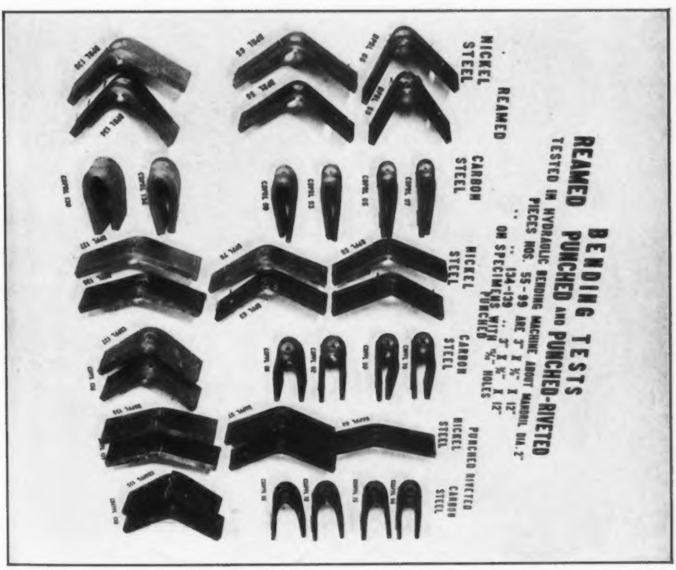


FIG. 2.—BENDING TESTS.

forced through by a plunger with its end rounded to any desired radius. The opening may be made of any desired width within certain limits. The plunger is attached to the piston of a hydraulic cylinder, consequently the pressure is very uniform. The piece is only partly bent, however, and is then flattened by the same machine.

For large, thick pieces a plunger 4½ in. in diameter is used, but ordinary structural material, up to 1 in. in thickness, is bent about a 2-in. plunger or mandrel.

The specimens for these tests were all 2 in. wide by 18 in. long, two being cut from the end of each plate, one edge of each specimen being the mill edge, finished in a universal mill, the other edge being planed. Through the carelessness of the machinist, several of the pieces were planed on both edges. One piece of nickel steel, *BL* 186, cut from the 1-in. plate, was lost, and a piece cut from the interior, marked *BL* 196, was substituted. All edges were smoothed by filing to remove burrs.

One-half of the total number of test pieces were first bent by the 2-in. mandrel, and afterward flattened until they cracked, or to 180° flat. This was done in order to find out how much bending the material would stand. All the carbon-steel specimens bent 180° flat, or until the capacity of the machine was reached, with a slight crack appearing in one piece only. The nickel steel also bent 180°, but cracked before the inner surfaces came in contact. With one exception, the nickel steel bent with a maximum radius of bend, such as would be formed about a mandrel having a diameter equal to 1.7 times the thickness of the material. One piece of angle material, *ABL* 61, was bent about the 4½-in. mandrel, but the bend was not so good.

The results are shown by the photograph, Fig. 1, Plate XV.

It seemed from these tests that the material would stand bending about a mandrel of a diameter equal to twice the thickness of the material without cracking; hence four of these mandrels were made, one for each thickness of material. If the material had bent to the diameters intended, probably the results would have been as expected, but the bending was too localized and an acute angle was formed, so that in no instance was the radius of the bend the same as that of the end of the mandrel. Cracks, therefore, were developed, the amount of bending being practically the same as before. One piece, *BL* 85, from the ½-in. plate, broke, and one-half was evidently thrown on the scrap pile and destroyed. The ¾-in. and 1-in. material did not show any cracks. In order to prove positively that the ⅜ and ½-in. plate material and the 1-in. angle material would stand a bending of 180° without cracking, with a diameter of inner surface of bend of $2t$ (t = thickness of material), the broken ends of some tensile test pieces were tried. This is considered a severe test, because of the straining, beyond the elastic limit, of all parts of the piece in tension. Many of the pieces were badly cut by the wedges.

The bending was done at Pencoyd under a small steam hammer. The load, however, was not applied to one place continuously, as in the Lukens tests, but was distributed over the middle of the piece, causing a gradually increasing curvature until the desired radius was obtained. One-half of the test pieces were bent to a diameter equal to three times the thickness and one-half to that of twice the thickness. Two pieces of ½-in. material broke because too sharp an angle was formed. These bends were very satisfactory, and proved that, by careful manipulation, almost any desired degree of bending could be obtained without fracture.

These results are shown in the photograph, Fig. 1, Plate XV.

It was not expected that the nickel steel would stand the same bending as the carbon steel, but it was believed that it would stand a sufficient amount to prove its toughness or lack of brittleness.

Wherever in the tables, in columns headed "Angle of bend," a number in parentheses appears, the unenclosed number is the angle to which the material bent when the crack first developed.

The angle of bend and the radius of the bend were obtained from sketches made by tracing carefully the outlines of the bent specimens. The radius was found by deducting, from the radius of the outside surface of the bend, the thickness of the specimen. The radius of the outside surface could be determined easily and with fair accuracy by being taken in a plane midway between the edges. Cracks usually developed first in this portion and spread to the edges. The testing was done in March and June, 1906.

BENDING TESTS, PUNCHED, REAMED, AND PUNCHED-RIVETED SPECIMENS.

The bending tests on nickel and carbon steels, like the corresponding tensile tests, were arranged to show relatively the effect of punching material full size, of sub-punching and reaming to full size, and of riveting punched work. The full-sized holes were $\frac{15}{16}$ in. in diameter and the sub-punched holes, $\frac{11}{16}$ in. in diameter, ⅛ in. of material, therefore, being reamed away.

The test was not included in the original scheme of testing, hence the specimens had to be cut from the excess material. None of the ⅝-in. plate of nickel steel was available, and only small pieces of the ½ and ¾-in. plates. For nine of the test pieces of nickel steel, it was necessary to use the pieces 4 by ½ by 12 in., designed and used for testing rivets in single shear. The greatest load on any joint was 63 400 lb., or 31 700 lb. per sq. in. This did not exceed the elastic limit, which from Table 30 was 62 000 lb. An examination of the results in Table 37 does not furnish any evidence that the previous use had worked an injury to the material. The location of the specimens in the plates seems to have had no influence on the results.

The specimens were machined to a width of 3 in. with these exceptions: Pieces *BPRL* 65, *BPPL* 78, *CBPRL* 65, *CBPPL* 78, and *CBAPPL* 75 were $2\frac{15}{16}$ in. wide. The punching, reaming, and riveting were conducted by the usual shop methods. The diameter of the die used in punching was $\frac{1}{16}$ in. larger than that of the punch. The same riveter as for the tensile tests, and the same scheme for protecting the specimens by a $\frac{1}{16}$-in. plate on each side, were used. The metal under the head of the rivet was bright, and, no doubt, was compressed as in the tensile-test specimens. As comparative results only were sought, there was no experimenting with mandrels of different diameters. The pieces were all bent, as described under the tests of plain specimens, in the testing room of the Lukens Iron and Steel Company, with a mandrel rounded on the end to a diameter of 2 in., a continuous pressure being slowly applied at the middle over the hole.

The edges of pieces and of holes were smoothed by filing. This was overlooked for the first three pieces tested, namely, *CBPRL* 65, 93, and 98; consequently, two pieces of nickel steel, *BPRL* 56 and 65, were tested in the same condition for comparison. The filing does not seem to have aided the carbon steel, but its effect on the nickel steel was marked. The two filed pieces bent 104° and 108°, respectively, and the unfiled pieces, 75° and 77°, with a corresponding difference in the radius of bend.

The results are given in Tables 37 and 38, and in the photograph, Fig. 2, Plate XV.

In every instance punching, sub-punching and reaming, and riveting, alter the $\frac{3}{4}$-in. material more than the $\frac{3}{8}$-in. material of both nickel steel and carbon steel. The $\frac{3}{4}$-in. plain specimens bent without cracking through the greater angle; but, in these tests, the reverse was true. The difference between the effects of punching and of reaming upon the $\frac{3}{8}$-in. nickel steel is greater than it is upon the carbon steel, but, upon the $\frac{3}{4}$-in. material, it is less.

Riveting increases the effect of the punching on both thicknesses of both steels and in about the same proportion. As a whole, these conclusions are the same as those drawn from the tensile tests. It could not be expected that nickel steel would bend through as great an angle as carbon steel. In the photograph, Fig. 2, Plate XV, the bent specimens are arranged side by side for comparison.

These tests were made in March and June, 1906.

Nicked Bending Test.

The following pieces, *NBL* 27, $2\frac{1}{2}$ by $\frac{3}{8}$ by 8 in., *NBL* 120, 2 by $\frac{3}{4}$ by 18 in., and *NBL* 189, 1 by 1 by 18 in., of both steels, were nicked $\frac{1}{8}$ in. and fractured in two places by bending. The fractures were uniform in appearance and free from laminations, pipe, or other defects. The fractures of the nickel steel were finer-grained than those of the carbon steel.

This determination of character of fracture was the sole object of the test.

Drifting Tests.

The consideration given this test, when the lay-out of tests was being prepared, was long and thorough. Specifications, for steel superstructures of all the large railroads, and of consulting engineers, were examined and tabulated. The specification of the drifting test seemed to be a purely arbitrary matter, as the amount of drifting a $\frac{13}{16}$-in. or a $\frac{15}{16}$-in. hole, punched full size and spaced $1\frac{1}{2}$ in. from a sheared or rolled edge, was required to stand without the edge of the plate or the periphery of the hole cracking, ran from 25 to 33%, 38%, 50%, and, in four instances, to 100 per cent. The other standard requirements of carbon steel for bridges are given later.

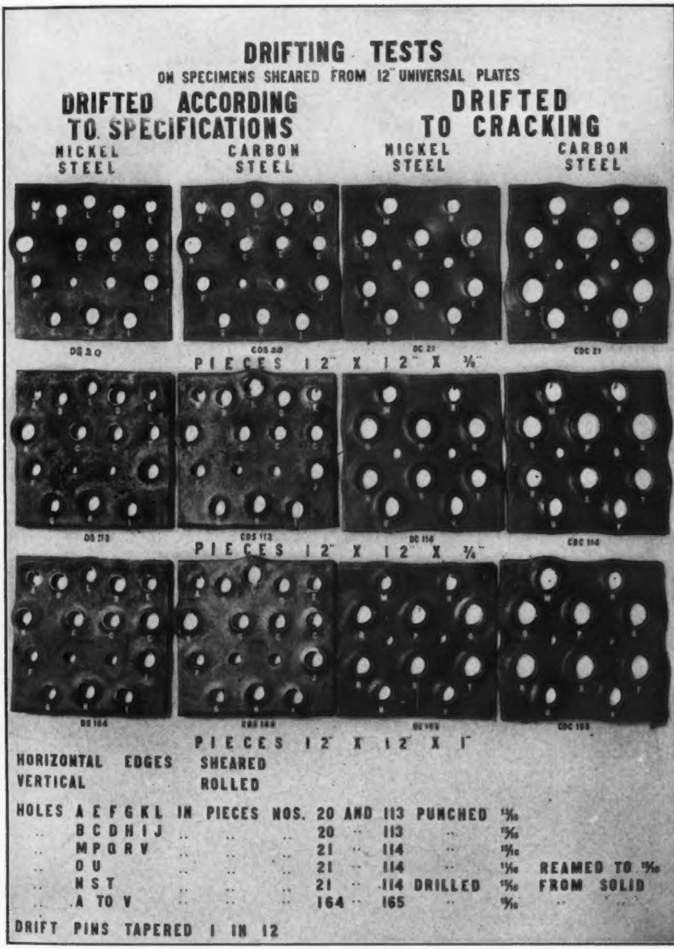
In the arrangement of holes finally decided upon, almost every recent specification, even the most severe, has been included. This explains the irregular grouping of holes. The number was limited by the size of the coupon selected, namely, 12 by 12 in.

With but two exceptions the hole required to be drifted was a punched hole—these two specified a $\frac{15}{16}$-in. hole reamed from a diameter of $\frac{11}{16}$ in. Holes of this sort were included, also holes drifted from the solid. The $\frac{3}{8}$-in., $\frac{3}{4}$-in., and 1-in. plates were selected, the first two because of the great bulk of structural material represented thereby, and the 1-in. plate, because it was proposed to use nickel-steel plates of this thickness in a large bridge, plans for which were being prepared.

Only one specification was found in which a taper for the drift-pin was specified, yet this is as important as the size and location of the hole. A pin with a large taper bulges the metal around the hole, and the edge cracks because of this, and not because the limit of enlargement has been reached. The ideal pin is one with a scarcely perceptible taper. A taper of 1 in 12 was adopted for this test, and several pins of this kind were made of specially hardened blue-chip and Sanderson steel.

The preparation of the test specimens and the testing were done at the Pencoyd Iron Works, the drifting being done by driving the pins with a heavy steam hammer, first through one side and then through the other, about $\frac{1}{2}$ in. at a time. In this way excessive bulging was prevented. It will be noted that the largest cracks and the greatest number of them are always on the side where the bulging is the greater. The holes were all carefully laid out, but, in punching, a slight shifting of some took place; the amount is not large enough to be of importance. Burrs were removed from the edges of the holes, but no filing was done. The diameter of the die used was $\frac{1}{16}$ in. larger than that of the punch.

Tables 39, 40, and 41, together with the photograph, Plate XVI, show the effect of drifting on specimens drifted according


DRIFTING TESTS
ON SPECIMENS SHEARED FROM 12" UNIVERSAL PLATES
DRIFTED ACCORDING TO SPECIFICATIONS
DRIFTED TO CRACKING
NICKEL STEEL
CARBON STEEL
NICKEL STEEL
CARBON STEEL
DS 20
CDS 20
DC 21
CDC 21
PIECES 12" X 12" X
DS 113
CDS 113
DC 114
CDC 114
PIECES 12" X 12" X
DS 164
CDC 165
PIECES 12" X 12" X 1"
HORIZONTAL EDGES SHEARED
VERTICAL ROLLED
HOLES A E F G K L IN PIECES NOS. 20 AND 113 PUNCHED
B C D H I J 20 113
M P Q R V 21 114
O U 21 114 REAMED TO
N S T 21 114 DRILLED FROM SOLID
A TO V 164 165
DRIFT PINS TAPERED 1 IN 12

to the requirements of the specifications mentioned, and Tables 42, 43, and 44 and the same photograph show the effect of carrying the drifting until cracks were developed at the edge of plate or the periphery of the hole. The nickel steel drifted with greater difficulty than the carbon steel. Through an oversight, Hole *K*, in *BS* 164, was not drifted as much as intended.

An examination of the tables shows that the carbon steel drifted better than the nickel steel. The difference is not marked in the tests made according to specifications; but in the tests to cracking, comparing the enlarged diameters of the holes, the $\frac{3}{8}$-in. nickel-steel plates drifted 58% as well, the $\frac{3}{4}$-in. material, 72% as well, and the 1-in. material, 67% as well as the carbon-steel plates, but the carbon steel is cracked to a larger extent than the nickel steel. It is probably fair to conclude that nickel steel will stand about 70% as much drifting as carbon steel, and satisfy the drifting requirements of many specifications written for medium steel. A hole spaced, as in the usual practice, from a sheared edge, will not stand drifting to the same extent as one spaced near a rolled edge. These tests are too few to give any conclusive figures, but probably the ratio is nearly as 1 to 2. Punched holes seem to drift as well as, if not better than, either reamed or drilled holes. The percentages giving extent of enlargement (Tables 42 and 43) may be arranged to show this as follows:

TABLE 19.—PERCENTAGE BY WHICH HOLES WERE ENLARGED.

Hole edge.	Material.	Punched.		Reamed.		Drilled.	
		Sheared.	Rolled.	Sheared.	Rolled.	Sheared.	Rolled.
3/8-in.	Nickel steel......	57 50	47 50	47	60	33	60
3/4-in.	" "	53 60	80 87	73	87	33	73
3/8-in.	Carbon steel.....	73 73	110 117	73	83	67	100
3/4-in.	" "	80 67	133 67	60	87	80	113

These tests were made in April and May, 1906.

CLOSE-PUNCHING TEST.

The close-punching test is rarely specified, because it furnishes but little information about the physical properties of material not obtained by other and more usual tests. It would show up brittleness or excessive hardness. In this series its value was in causing to be punched at one time, a sufficient number of holes in the nickel steel of different thicknesses to permit forming an idea of the effect on the machine and punches, and also to show the ability of the metal to stand close punching.

A rather light punching machine in the shop at Pencoyd was used; and, while no part gave way, the $\frac{3}{4}$-in. material was too heavy for it. Twenty-eight $\frac{13}{16}$-in. holes, and twenty-five $\frac{15}{16}$-in. holes were punched in each of the two pieces, *CP* 22 (12 by 12 by $\frac{3}{8}$ in.), and *CP* 115 (12 by 12 by $\frac{3}{4}$ in.), and about the same number in the corresponding carbon-steel pieces. The arrangement of these holes and the results of the test are shown in detail on Fig. 1, Plate XVII.

Two punches were broken, one by stripping and one by shearing, while punching the nickel steel. The steel in these punches was considered exceptionally good, but the toolmaker was not familiar with it, and frequent breakages had happened in regular work, consequently, the blame cannot be laid altogether upon the nickel steel. At the time, the shop management regarded this work as more of a test for the punches than for the nickel steel. Before this, in connection with this investigation, considerable punching of nickel steel had been done at Pencoyd, with but one breakage, a $\frac{15}{16}$-in. punch. One punch and one die were broken in punching material for struts at Ambridge. Several holes, $\frac{11}{16}$ in. in diameter, were punched in $\frac{3}{4}$-in. nickel-steel plates, both in preparing test specimens and in preliminary experimental work. As regards the effect upon the punch, this is not a serious matter. The heavier punching machines now in the large shops would not be over-strained were nickel steel to be introduced, but the smaller ones unquestionably would be.

The rapidity of punching would not differ from present practice were nickel steel introduced, though, at the outset it might, because the "punching" flies out with considerable force and with a loud sharp report, and men unaccustomed to this think there is danger.

The punched hole in nickel steel is very different from that in carbon steel—its edges are clean and smooth, with no appreciable compression of the "punching." For this reason it is thought by some that punching is less injurious to nickel steel than to carbon steel. A discussion of this will be found under tensile tests and bending tests of punched specimens.

The holes in the "close-punching" coupons at the left and top edge were spaced so that $\frac{1}{8}$ in. of metal was left between the edges of adjacent holes, $\frac{3}{16}$ in. between the edge of the hole and the rolled edge, and $\frac{1}{4}$ in. between the edge of the hole and the sheared edge.

The diameter of the die used was $\frac{1}{16}$ in. larger than the punch.

The partitions between the holes actually varied from $\frac{1}{16}$ to $\frac{1}{8}$ in., and between the hole and the sheared and rolled edges from $\frac{3}{16}$ to $\frac{5}{16}$ in. The actual distances are so much smaller than stipulated in any specification ever written that a variation of $\frac{1}{16}$ in. or more is unimportant.

The photograph shows the die side of these plates.

The effect of the punching is practically the same for both steels. Where the partitions between the holes in the nickel-steel plates are

PLATE XVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXIII, No. 1103.
WADDELL ON
NICKEL STEEL FOR BRIDGES.

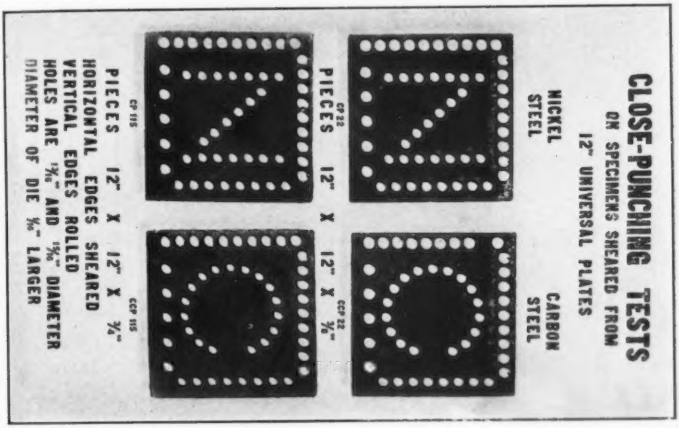


Fig. 1.—Close-Punching Tests.

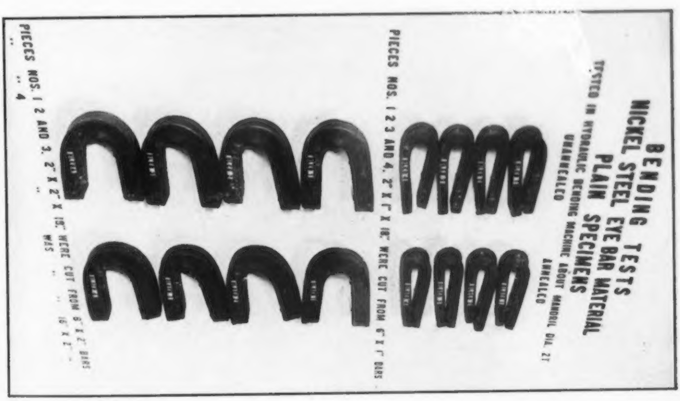


Fig. 2.—Eye-Bar Material Tests.

shown broken down, this resulted from inaccurate punching; as explained, the operator was nervous. The deformation of each hole by the succeeding one is alike in both steels, it occurs on the die side only, all holes on the punch side being truly-circular in form; there is, however, a slight depression of the walls between adjacent holes. There was no deformation of the metal between the other holes in these plates. This test was made in March, 1906.

Hammering-Flat Tests.

A clause covering the hammering-flat test can be found in one or two specifications. It is strictly a punishment test to detect brittleness. Material is rarely treated in this way in bridge-shop practice except in fillers; but, in riveted-steel pipe manufacture, the corners of many plates must be scarfed by cold hammering.

The test was made on 2 by 8-in. pieces, marked *HL* 14 and *HL* 131, cut from the $\frac{3}{8}$-in. and the $\frac{3}{4}$-in. plates, respectively, of each steel, under a light steam hammer. A small round was first used to localize the distortion, but only for a few blows, most of the work being done by hammering the end held on the anvil. There was very little difference between the nickel steel and the carbon steel; the latter broadened out more, but the thickness at the edge was not less and the cracks were a little larger. Nickel steel is so stiff under treatment of this kind that the strain on the hammer is very great. The results are shown in Table 20.

TABLE 20.—Hammering-Flat Tests.

Material.	Original.		Final.		Remarks.
	Width, in inches.	Thickness, in inches.	Width, in inches.	Thickness, in inches.	
Nickel steel......	2	$\frac{3}{8}$	$2\frac{17}{32}$	$\frac{1}{32}$	No cracks developed.
" "	2	$\frac{3}{4}$	$2\frac{9}{16}$	$\frac{5}{64}$	Two small cracks started.
Carbon steel.....	2	$\frac{3}{8}$	$2\frac{11}{16}$	$\frac{1}{32}$	Two small cracks started.
" "	2	$\frac{3}{4}$	$2\frac{15}{16}$	$\frac{3}{32}$	One quite large crack.

This test was made in the forge shop, at Pencoyd, in March, 1906.

Shop-Tooling Tests.

By shop-tooling tests is meant submitting the steel to all the shop operations necessary to its fabrication, and watching closely the results.

Sawing.—It has been stated that the 8 by 8 by 1-in. angle was sawed across. A time test was not made. The saw was not injured. Probably it revolved at the usual rate, but was fed more slowly.

Shearing.—All the usual sizes of plates may be safely sheared in any of the large shops—in these tests the 12 by 1-in. and a 36 by $\frac{7}{8}$-in. were the heaviest sections sheared. The strain on the shears is considerable, and probably, were nickel steel introduced, heavier machines

would have to be built. The same time is required, whether nickel steel or carbon steel is sheared.

Punching.—A discussion of punching was introduced under the close-punching test. No difficulties greater than getting the operators accustomed to the material are presented. There would be a slight increase at the outset in the breakages of punches, but, without doubt, the toolmakers would meet this extra demand upon them.

Reaming.—Progressive shops, to-day, are adopting dry reaming with high-speed, steel tools, the only lubricant used being a graphite paste. Many shops, however, still retain the slow-speed, ordinary, steel drill with an excessive use of a lubricant, principally water. This sort of tool will not stand up with nickel steel: it over-heats, the edge crumbles, the tool binds, and the progress is painfully slow. With the special high-speed, steel tool revolving at great rapidity, nickel steel may be reamed at the same rate as carbon steel; graphite is used more liberally, and the tool wears out more frequently. In the fabrication of the struts for compression tests, nickel steel and carbon steel were being run through a drill press, side by side. The behavior of the tools was carefully watched, though no time test was made. Some of the operators, Hungarians and Poles, had trouble with the sticking of the tool, but not after they became accustomed to the material.

Drilling.—A number of holes were drilled in specimens for drifting tests, riveted-joint tests, etc., and a special time test was also made. The general observations made under reaming might be repeated here. Ordinary steel tools will not last for any length of time, but the special steels now in common use may be used successfully.

A piece cut from the 1-in. plates of nickel steel, marked *SD* 161, and one from the 1-in. plate of carbon steel, marked *CSD* 161, were used for the comparative test, with the results shown in Table 21.

TABLE 21.—DRILLING TESTS, WITH COMMON $\frac{15}{16}$-INCH TWIST DRILL.

MEDIUM CARBON STEEL.

No. of hole.	Depth of hole, in inches.	Time consumed.	Speed, in revolutions per minute.	Remarks.
1	1	1 min. 17 sec.	144	Drill was not altered.
2	1	1 " 18 "	144	" " " "

STRUCTURAL NICKEL STEEL.

No. of hole.	Depth of hole, in inches.	Time consumed.	Speed, in revolutions per minute.	Remarks.
1	⅝	1 min. 10 sec.	144	Drill was worn out.
2	⅞	1 " 26 "	144	" " " "

The ordinary drill will stand up several hours with the usual carbon steel; with nickel steel, it gets excessively hot, turns blue, the point crumbles, and the clearance wears off.

In the drilling of Hole No. 2, after 56 sec., the drill was permitted to cool off before finishing the hole. Such a tool, however, would not be used in practice.

TABLE 22.—DRILLING TESTS, WITH CLEVELAND HIGH-SPEED NO. 10 STEEL (BLUE-CHIP) TOOL.

MEDIUM CARBON STEEL.

No. of hole.	Depth of hole, in inches.	Time consumed.	Speed, in revolutions per minute.	Remarks.
1	1	0 min. 51 sec.	210	Same drill used throughout—it was in perfect condition.
2	1	0 " 49 "	210	
3	1	0 " 49 "	210	
4	1	49 "	210	

STRUCTURAL NICKEL STEEL.

No. of hole.	Depth of hole, in inches.	Time consumed.	Speed, in revolutions per minute.	Remarks.
1	1	1 min. 16 sec.	144	Same drill used in above test, after being ground, was used throughout—the point had begun to crumble at end of second hole and was in very poor condition at end of fourth hole.
2	1	1 " 16 "	144	
3	1	1 " 14 "	144	
4	1	1 " 15 "	144	

The speed was changed for the nickel steel, because of the danger of breaking the drill at the higher speed. The carbon-steel chips were steel gray, and the nickel steel, blue. No lubrication was used.

The speed, therefore, with which the carbon steel was drilled was 1 in. in 35 sec., and the nickel steel, 1 in. in 52 sec. A blue-chip tool, in ordinary work, would last half a day without sharpening, whereas, if used for 5 or 6 min. on nickel steel, it is necessary to sharpen it. This factor is the more important by far, as considerable time is wasted in the re-dressing.

This test was made in the machine shop at Pencoyd, in September, 1906.

To obtain a comparison of all factors, a much longer test would be necessary; the foregoing, however, throws some light on the subject.

Planing.—For planing, the edge planer at the Pencoyd plant was used, and a blue-chip steel cutter. An attempt was first made to determine the depth of cut that could be made in the 1-in. plate material of nickel steel and of carbon steel. Coupons 4 in. wide and 18 in. long were bolted rigidly to the planer bed. With the carbon steel it was possible to make a cut of $\frac{1}{8}$ in., but the belt could not furnish sufficient power to cut this depth in the nickel-steel plate. A $\frac{1}{16}$-in. cut was made, and, after a total depth of $\frac{5}{16}$ in. had been removed from the 18-in. piece, the edge was burned off the tool.

Successive attempts proved it possible to take a maximum depth of about $\frac{3}{32}$ in. of cut in the nickel steel; the surface, however, was

very rough and torn. Eight cuts, $\frac{1}{16}$ in. in depth, were next taken from each of the 18-in. plates, and the times noted, as follows:

	Time, in seconds.								
Carbon steel	$6\frac{1}{4}$	$5\frac{3}{4}$	6	$5\frac{3}{4}$	6	$5\frac{3}{4}$	$5\frac{1}{2}$	$5\frac{3}{4}$	Average $5\frac{7}{8}$
Nickel steel	6	6	$5\frac{3}{4}$	6	6	$6\frac{1}{4}$	$5\frac{3}{4}$	$5\frac{1}{2}$	Average $5\frac{7}{8}$

The carbòn steel apparently had no effect on the tool. The nickel steel, after eleven cuts, had burned the edge off. A cut of $\frac{1}{32}$ in. would be probably the maximum depth that would be taken from a long plate, because of the destructive effect on the tool.

It may be concluded, therefore, that nickel steel is more difficult to machine than carbon steel, and the amount of work done in a given time would be less, probably not more than half as much. The strips from the carbon steel were short in length, and steel gray in color; the strips from the nickel steel were longer, and deep blue in color. This is evidence of the greater power necessary to cut the nickel steel. This test was made in March, 1906.

Pneumatic Chipping.—Pieces 4 by $\frac{1}{2}$ in. were bolted securely by two bolts to a heavy I-beam. The workman was a regular chipper; he operated a Boyer hammer, size $1\frac{1}{16}$ by 3 in.; and the chisels were of standard size. The results are shown in Table 23.

TABLE 23.—RESULTS OF PNEUMATIC CHIPPING.

	NICKEL STEEL.		CARBON STEEL.	
Cuts.	Time required.	Length of strips.	Time required.	Length of strips.
First cut	3 min. 9 sec.	$16\frac{3}{8}$ in.	3 min. 53 sec.	$16\frac{3}{8}$ in.
Second cut	5 " 6 "	$13\frac{1}{2}$ "	2 " 18 "	$13\frac{1}{2}$ "
Third cut	3 " 15 "	$12\frac{1}{2}$ "	1 " 35 "	$12\frac{1}{2}$ "
Fourth cut	4 " 3 "	$11\frac{3}{4}$ "	2 " 50 "	$11\frac{3}{4}$ "
Totals	15 min. 33 sec.	$54\frac{1}{8}$ in.	9 min. 56 sec.	$54\frac{1}{8}$ in.
The first two cuts in the nickel steel were made by a workman using a short chisel. The third and fourth cuts in the nickel steel were made by another man with a new chisel taken from the stock room. (Workmen do not like these chisels because of their length; they think they cannot do as much work with them as with the short ones.) Material removed=2.58 cu. in.			All cuts made with new chisels. Chisel was not affected. Material removed=3.29 cu. in.	

Material.	Time.	Length of cut.	Amount removed.
Nickel steel	15.55 min.	54.12 in.	2.58 cu. in.
Carbon steel	9.92 "	54.12 "	3.29 " "
ON A BASIS OF 10 MIN.			
Nickel steel	10 min.	34.8 in. (64%)	1.66 cu. in. (50%)
Carbon steel	10 "	54.6 " (100%)	3.32 " " (100%)

Hand Chipping.—Pieces, 4 by ½ by 18 in., were bolted in the same manner as for the pneumatic-chipping test. The workman was an expert chipper of long experience. The results are shown in Table 24.

TABLE 24.—Results of Hand Chipping.

Nickel Steel.			Carbon Steel.		
Cuts.	Time required.	Length of strips.	Cuts.	Time required.	Length of strips.
First cut...	5 min. 0 sec.	5¼ in.	First and second cut...	5 min. 0 sec.	8 in.
Workmen rested 5 min.			Workmen rested 5 min.		
Second cut.	5 min. 0 sec.	6 in.		5 min. 0 sec.	7 in.
	3 " 22 "	2⅛ "	First cut was 10⅛ in. long, remainder was on second cut.		
	1 " 4 "	1¼ "	Second and third cuts	4 min. 0 sec.	5⅛ in.
	1 " 32 "	1¼ "	⅝ in. was on second cut, and 4½ in. on third cut.		
Totals......	15 min. 58 sec.	16¼ in.	Totals..................	14 min. 0 sec.	20⅛ in.
In chipping the second strip, the chisel was broken after 2 min. 32 sec.; chipping ⅛ in. further turned edge of chisel. After 3 min. 22 sec. of chipping, new chisel was tried; in 1 min. 4 sec., piece was broken out of chisel, and, after another 1 min. 32 sec., a piece was broken out of a second new chisel.			Chisel was not affected.		
Material removed = 0.97 cu. in.			Material removed = 1.24 cu. in.		

Material.	Time.	Length of cut.	Amount removed.
Nickel steel............	15.97 min.	16.25 in.	0.97 cu. in.
Carbon steel............	14.0 "	20.12 "	1.24 " "
On a Basis of 10 Min.			
Nickel steel............	10 min.	10.2 in. (71%)	0.61 cu. in. (69%)
Carbon steel............	10 "	14.4 " (100%)	0.89 " " (100%)

All this work was done in the shop at Pencoyd, and, though the period of testing was short, a fair comparison may be drawn. It may be assumed that the time required to chip nickel steel would be one-half longer than for the usual carbon steel.

These tests cover practically all the shop operations except milling, and, as this is similar to planing, the same conclusions may be drawn.

It is not possible from these few tests to state even roughly what the additional cost of fabrication would be, should nickel steel be introduced.

With the addition of one or two machines and a few men, the same number of pieces of nickel steel as of carbon steel could be handled in the shop, provided sections of the same size were used in the structures; but if advantage were taken of the high tensile strength of nickel steel, to cut down the size of the sections, then the output in tons would be decreased very considerably without a proportionate decrease in operating expenses. The general expenses of sales and auditing departments, of general office, drafting-room, template shop, and, in the bridge shop itself, of laying-out, assembling, riveting, painting, and dead labor would not be affected materially.

Bearing-on-Pins Test.

The bearing-on-pins test has considerable value for comparative purposes. It was designed to give the bearing values of structural nickel steel and carbon steel and of rivet nickel steel and carbon steel on a hard steel pin 1 in. in diameter. A special contrivance, illustrated by Fig. 73, was built for this purpose.

This contrivance was also designed to test rivets in double shear, which explains the presence of a ⅞-in. hole over the 1-in. hole. The steel pin was turned to a driving fit, and was supported at four points to minimize bending. The two inner bearings are of tool steel, specially hardened and spaced with just sufficient clearance to permit easy withdrawal of the specimen after testing.

The pieces tested were cut from the 12 by 1-in. Universal rolled plates of nickel steel, Heat No. 17 673, and of carbon steel, Heat No. 33 342, three from each; also three pieces from each of two similar plates rolled from a rivet grade of nickel steel and one of carbon steel. The following physical properties of the rivet material were reported by the mill:

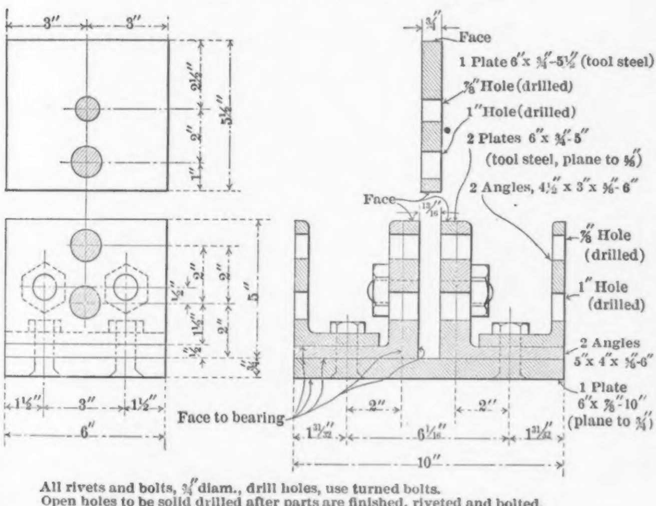
Composition	Nickel rivet steel. Heat No. 2 096.	Carbon rivet steel. Heat No. 19 241.
Nickel	3.30	....
Carbon	0.17	0.13
Manganese	0.53	0.43
Sulphur	0.028	0.025
Phosphorus	0.012	0.028
Silicon	0.020	.....
Ultimate strength	84 200 lb. per sq. in.	54 100 lb. per sq. in.
Elastic limit	51 800 " " " "	37 600 " " " "
Elongation in 8 in.	22.5%	30.5%
Reduction of area	50.3%	59.9%

The twelve specimens were each 3.83 in. wide, 4.50 in. long, and of the thickness of the material from which they were cut. Slight variations in length were neglected, because they would not affect the read-

ings materially. The average distance from the top of the specimen to the top of the pin-hole was 3.83 in., but varied from 3.73 to 3.86 in.

All testing was done in the testing room of Tinius Olsen and Company. The load was applied by the movable head of a 100 000-lb. machine, and was read on the beam in the usual way; the compression was measured by a compressometer, reading directly to $\frac{1}{10\,000}$ in., this delicate adjustment being effected by an electric contact ringing a tap bell.* The zero readings were taken under a load of 2 000 lb., in order that all clearances in the contrivance should be eliminated from the results.

CONTRIVANCE FOR MAKING TESTS OF RIVETS IN DOUBLE SHEAR AND FOR DETERMINING BEARING VALUES OF NICKEL AND CARBON STEELS ON HARD PINS.



All rivets and bolts, ¾" diam., drill holes, use turned bolts.
Open holes to be solid drilled after parts are finished, riveted and bolted.
Tool-steel will be furnished to shop.
Other material to be taken from stock.
Use no paint in assembling.
Face backs of angles accurately at right angles to a thickness of ½".
Temper tool-steel after holes are drilled.

FIG. 73.

Two projecting arms rested on the base of the apparatus and two against the under side of the specimens which projected below the pin for this purpose. Under each increment of load there was a yielding of the pin and supports, as well as a compression in the specimen above the pin, and, after the yield point had been reached, a distortion of

* A description of this instrument may be found in Olsen and Company's Catalogue, Part C, page 16.

the pin-hole. This yielding varied with each test; the pin was always inserted in the same position. To determine accurately the amount of yielding of pin and supports, a piece of nickel steel with a half-hole only was used; one pair of arms, instead of resting against the specimen, rested against the under side of the pin. A number of such readings were taken before, between the specimen tests, and after. The pin, after the series of tests on the plate material was made, was found to have been bent perceptibly, hence a second pin was used for the tests of rivet material.

The half-hole piece of carbon steel was used only once, because it was so badly distorted; and the readings, therefore, were discarded.

Readings Nos. 1 and 5 were neglected, because so large a part of the deformation of pin and contrivance caused by this first loading became permanent that distorted readings would have resulted from their use. The readings are not accurate to the last figure. Readings Nos. 3 and 4 (Pin 1) and Nos. 6 and 7 (Pin 2), taken consecutively, show that the probable error through inaccuracies of readings is very small.

It will be noticed that after each loading there was an increase in the permanent deformation of the contrivance; therefore, in making the correction to the intermediate tests of specimens, this was taken into consideration. Reading No. 8 (Pin 2) shows that for some reason the conditions were changed, and that this is not the result of inaccurate micrometer observations is shown by the readings for Specimen 3*N*. The conditions became again more nearly normal for 3*C*.

It was impossible with the apparatus at hand to make corrections for every variable. The results of these tests are not scientifically exact, but for all practical purposes they are closely approximate. The corrected amounts of compression for each specimen were obtained by deducting, from the micrometer readings for the specimens, the micrometer readings on the half-hole pieces, Nos. 1 to 9, inclusive, in the following way: The plate carbon-steel readings are based on No. 2 (Pin 1); the nickel steel on intermediate values between No. 2, and an average of Nos. 3 and 4. The rivet steels, 1*N* and 1*C*, on averages of Nos. 6 and 7; 2*N* and 2*C* on intermediate values between averages of Nos. 6, 7, and 8; 3*N* and 3*C* on intermediate values between Nos. 8 and 9.

Errors arising from this method affect only the amounts of compression and not the elastic limit, which was obtained directly from curves plotted from the original readings. The loading of the carbon steel was not carried as far as the plate nickel steel, because of the very large resulting compressions in the specimens, the curves becoming almost horizontal. The elastic limit is fairly well marked except for Curve *BRP* 116.

Tables 45 to 49, inclusive, give in detail all the readings taken and also the corrected readings.

Table 50 is arranged to show the essential features of each test for easy comparison. The figures in parentheses in the column headed "Elastic Limit" are those obtained from the corrected readings. As shown in the column of "Original Dimensions of Pin-Hole," the pin-holes in the plate specimens were 1 in. in diameter, while those in the rivet specimens were slightly enlarged. An examination of the readings of the plate steel revealed variations for loadings under 50 000 lb. not directly traceable to the contrivance—they were greater for the nickel steel. The pin-holes in these specimens were not left perfectly smooth by the drill. In the case of the carbon steel, this surface, after a small load, became as polished as glass, but the surface of the nickel steel still showed roughnesses, even after the maximum load had been applied. The entire surface, therefore, could not have had a bearing, and as the bearing surface varied, it could be supposed that the amount of compression over the pin, under equal loads, varied also, and that the extent of this variation would decrease as the load increased. The pin-holes of the rivet steels, therefore, were scraped and smoothed with emery.

It is evident from Table 50 that the nickel steel has a very much higher bearing value than the carbon steel; both within the assumed elastic limit and beyond, the resistance to compression becoming greater relatively as the load increases. Under a load of 115 000 lb. per sq. in., the amount of compression is only one-tenth of what it is in the corresponding structural carbon steel.

These tests were made in February and March, 1906.

Bending on Pins.

Specimens 1 in. square and 18 in. long were used. The surface of the plate on two sides was left and the two others were machined. On one machine surface the pieces rested on two supports 12 in. apart—the points of bearing being rounded hard steel. The load was applied at the middle through a steel casting, with the edge rounded to a radius of $\frac{1}{4}$ in. and bolted to the movable head of a 100 000-lb. Olsen testing machine having an autographic attachment. The results were obtained from the pencil record. The movement of head within the elastic limit was at the rate of 1 in. in 12 min., and beyond this point 1 in. in 3 min. After the maximum load had been reached, the test was discontinued.

The results are given in Table 25.

The depth of the carbon-steel specimens being slightly less than those of the nickel steel, to make the results strictly comparable, the elastic limit of the carbon steel should be increased by 25 lb. and the maximum load by 50 lb.

The values for "elastic limit" in Table 25 are those obtained at the point at which the stress-strain ratio changes, as indicated by a deviation of the record from a straight line. If this point be determined

as was the yield point in the tensile tests, it would be 5 100 lb. for the nickel steel, and 2 570 lb. for the carbon steel, and for the elastic ratios, 54 and 50, and the deflections, 0.101 in. and 0.066 in., respectively.

TABLE 25.—BENDING-ON-PINS TESTS.—STRUCTURAL NICKEL STEEL AND CARBON STEEL.

Specimens, 1 by 1 by 18 in. — Supports, 12 in. between centers.

Mark.	Load per Square Inch.			Deflection at elastic limit, in inches.
	Elastic limit, in pounds.	Maximum load, in pounds.	Elastic ratio. Percentage.	
Nickel-steel, 12-in. Universal Plate—Heat No. 17 673.				
*BNP*156	4 740	9 580	50	0.090
*BNP*157	4 520	9 450	48	0.080
*BNP*159	4 640	9 480	49	0.075
Average	4 630	9 500	49	0.082
Carbon-steel, 12-in. Universal Plate—Heat No. 33 342.				
CBNP 156	2 240	5 100	44	0.044
CBNP 157	2 460	5 220	47	0.055
CBNP 159	2 490	5 080	49	0.060
Average	2 400	5 130	47	0.053

The general conclusions would not be altered, however, the elastic limit for the nickel steel being nearly double that for the carbon steel, with a deflection only 1.55 times as great. As in the "bearing-on-pins" test, therefore, it would seem that the nickel steel was decidedly the stiffer material. The maximum load carried by the nickel steel was 1.85 times that sustained by the carbon steel.

COMPRESSION TESTS OF STRUTS.

Twelve struts were designed for this test, three of nickel steel, 10 ft. long, and three of same, 30 ft. long from center to center of end pin-holes, and the same number of each length of medium carbon steel.

Each was built of four angles, 3 by 3 by ⅜ in., and two plates, 12 by ⅜ in., laced on two sides with bars 2½ in. and ⅜ in., and had an area of cross-section of 17.44 sq. in. The ends were heavily reinforced to insure failure in the body of the strut.

The design of these struts is shown on Plates XVIII and XIX. The material was rolled by the Carnegie Steel Company, at its Homestead Works, and was fabricated in the shops of the American Bridge Company, at Ambridge, Pa. The tests were made on the 2 160 000-lb. hydraulic testing machine of The Phœnix Iron Company, at Phœnixville, Pa.

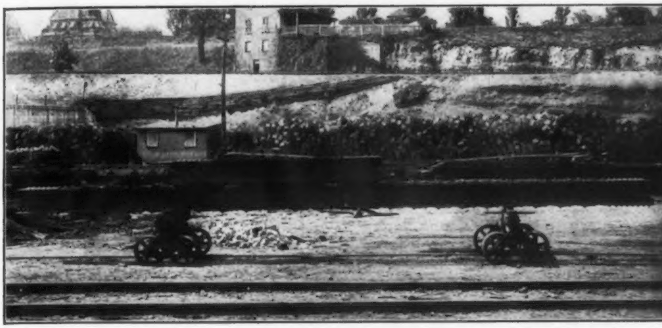


FIG. 1.—LONG-COLUMN TESTS.

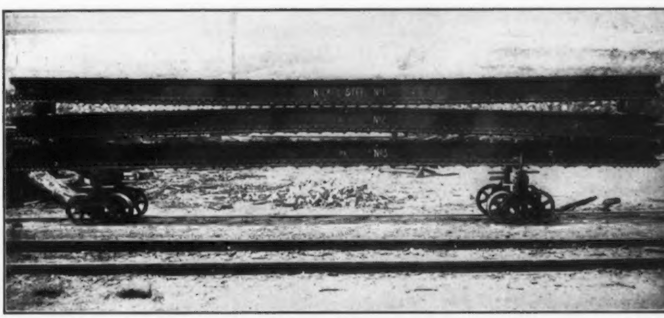


FIG. 2.—LONG-COLUMN TESTS.

FIG. 3.—LONG-COLUMN TESTS.

It was intended that all material of each kind, except the rivet steel, should be from the same melt. This was not effected; but the record in Table 26 shows that the variations are not greater than would occur in any bridge member.

TABLE 26.—MATERIAL USED IN THE FABRICATION OF STRUTS.

	Nickel steel, Heat No.	Carbon steel, Heat No.
Side plates, 12 by 3/8 in	17 673	41 520
Angles, 3 by 3 by 3/8 in	17 749 (1-10 ft. 17 065)	"
Batten plates, 16 by 3/8 in	2 116 (23)	" (1) 45 254
Pin plates, 12 by 3/8 in	17 673	"
" " 11 1/4 by 3/8 in	17 749	"
" " 6 by 3/8 in	17 673	"
Lace bars, 2 1/2 by 3/8 in	17 749 (55)	" (6) 60 161
Rivets, 7/8 in	2 192	Stock.

COMPOSITION.

Heat No.	Nickel.	Carbon.	Manganese.	Sulphur.	Phosphorus.	Silicon.
17 673	3.68	0.41	0.76	0.020	0.005	0.046
17 749	3.52	0.36	0.76	0.030	0.012	0.058
2 116	3.50	0.38	0.70	0.04	0.03	0.04
17 065	4.25	0.463	0.67	0.014	0.019	......
2 192	3.28	0.20	0.62	0.023	0.010	......
41 520	none	0.23	0.56	0.025	0.025	0.020
45 254	none	......	......	......	0.028	......
60 161	none	0.19	0.45	0.030	0.005	......

PHYSICAL PROPERTIES.

Heat No.	Elastic limit.	Ultimate strength.	Elongation in 8 in.	Reduction of area.
17 673	67 500	112 600	16.5	50.0
17 749	......	....	......	......
2 116	58 900	92 800	19.5	28.4
17 065	68 500	108 600	18.75	41.3
2 192	56 300	77 600	25.50	51.3
41 520	44 200	66 200	29.25	56.2
45 254	37 100	61 300	31.75	56.2
60 161	40 700	60 800	30.25	60.8

The bending tests on this material were very satisfactory. Those for the nickel steel are fully detailed elsewhere in this paper. Those for the carbon steel, Heat No. 41 520, bent 180° with an opening of ¼ in. before cracks showed. Heat No. 45 254 bent 180° flat without cracking.

The material was received at the shop during January and February, 1905. Shopwork was proceeded with during the latter part of April.

"De Pontibus" specifications for medium-carbon steel were to govern the fabrication, the detail drawing, however, was not so marked, so the punching was started full size. The 6-in. pin-plates for all the struts, and twelve holes in one 10-ft. nickel-steel angle and three holes in another, were punched $\frac{15}{16}$ in. in diameter instead of $\frac{13}{16}$ in. The shop also mis-sheared the nickel-steel pin-plates. These errors, indirectly, caused a delay of one year; and, before the rejected material could be replaced, all work was ordered stopped. The shop was ready to resume in March, 1906, but, during the interval, the rivet rounds, one 10-ft. nickel-steel angle, and several lace-bars were lost. In order to proceed, a 3 by 3 by $\frac{3}{8}$-in. angle from Heat No. 17 065 was used, and two carbon-steel lace-bars were substituted for each missing nickel-steel bar. These were placed at the ends of the struts so as to have a minimum effect on the results. The 6-in. plates of nickel steel were replaced, and the other mis-punched material was used. Additional nickel-steel rivet material was ordered. Carbon-steel rivets were taken from stock. These matters being finally adjusted, shopwork was again taken up in April, and pushed through without mishap. Extreme precautions were taken at every step to keep the two steels separate.

One punch and one die were broken during the punching of the nickel steel. All reaming was done dry, with only graphite lubricant, and with high-speed blue-chip reamers. One reamer could ream to $\frac{15}{16}$-in. about 60 in. of $\frac{13}{16}$-in. holes in the nickel steel before needing regrinding; while, in the carbon steel, it could do four or five times as much. The riveting was done with a machine capable of exerting a pressure of about 35 tons per sq. in.

This riveting work was all done at one time, on the same machines, the rivets being heated in the same fire. No variation was made from regular shop practice. All rivets were tested, but none cut out. The heads of many of the nickel-steel rivets cracked at the edge in making and in driving, but, on the whole, the appearance was very good.

The struts were shipped to Phœnixville on April 21st, where they were tested early in June.

A special cast-steel bolster was made by the Phœnix Iron Company for holding the ends of the struts in position in the machine. The struts, therefore, were free to move vertically, and, in this direction, were the same as round ended, this being the usual condition in pin-connected bridge construction. The short struts were not supported, except at the ends; but the long struts, in order to counteract the bending from their own weight, were counterweighed at two intermediate points, 8 ft. apart. To the eye, these long columns, when in the machine, appeared straight, and a stretched string showed a camber upward not exceeding in any instance more than 0.34 in., and two struts that showed such a camber failed by bending in the opposite direction. It would seem, therefore, that this initial flexure had no influence on the result.

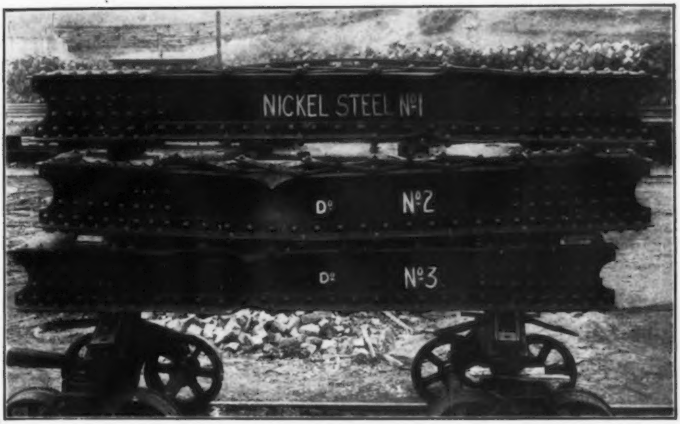


FIG. 1.—SHORT-COLUMN TESTS.

FIG. 2.—SHORT-COLUMN TESTS.

As the load was applied to these long columns, flexure took place, the normal condition being resumed on the removal of the load until a permanent set was evident.

The pressure was registered by a Shaw mercury gauge, graduated to 10 lb., equivalent to 32 000 lb. load on the strut. Intermediate values could be easily read. The accuracy of this gauge was examined and certified to by the maker.

The amount of compression in the struts under each increment of load was measured by a contrivance, designed by Tinius Olsen and Company, consisting of a long arm (7.54 ft. for short struts and 27 ft. for long struts), one end resting immovably on one batten-plate, and the other carrying a pointer resting in a punch mark made in the batten-plate at the other end of the strut and turning on an axis, placed so that any variation in the length of the strut was multiplied five times and measured on an arc divided to hundredths of an inch. This becomes a very delicate apparatus, when the length between bearing points is considered. The movement of the head of the machine varied from $\frac{1}{16}$ to $\frac{3}{8}$ in. per min.

Strut No. 1, the first tested, was the short one of nickel steel, having the $\frac{7}{16}$-in. angle substituted from a heat containing $4\frac{1}{4}\%$ of nickel. The load was applied continuously, and the pointer of the gauge and the mercury column were watched carefully. This procedure was adopted for the first test, with some hopes that the elastic limit could be detected by the movement of the pointer or by the mercury column, or perhaps by both. There was no perceptible change in the rate of movement of either.

The two mis-punched angles were also used in this strut, also carbon-steel lace-bars to replace the lost ones of nickel steel, as described before.

In succeeding tests, loads increasing in amounts were alternately applied and removed, and the movement of pointer noted. This method was very successful in establishing what loads produced a permanent set, also the amount of temporary shortening under these loads. This quantity is the sum of two changes, one resulting from a compression of the material and the other from the bending of the strut. It was thought that a succession of applications of loads in this way would weaken the strut below what it might be expected to carry, so that, in the beginning of the work, a greater difference between the successive loads was adopted. The number was gradually increased, as the maximum strength of the strut did not seem to be affected thereby.

In Table 27 the smallest appreciable permanent set was assumed as 0.005 in. and as a "set" of 0.005 in. in 7.54 ft. corresponds to a set of 0.018 in. in 27.0 ft., the load producing approximately this amount, 0.015 in., is included for comparison.

It will be noticed that in one instance only did a long strut of nickel steel have a set of this amount.

TABLE 27.—SUMMARY OF RESULTS OF COMPRESSION TESTS OF STRUTS.

Strut No.	Nickel Steel.			Carbon Steel.				
	Load per square inch producing:			Load per square inch producing:			Ratio: Nickel steel / Carbon steel	
	Set = 0.005 in.	Set = 0.015 in.	Failure.	Set = 0.005 in.	Set = 0.015 in.	Failure.	Set = 0.005 in.	Failure.
10-Foot Struts.								
1.........	53 700	....	68 500	29 600	....	38 900	....	....
2.........	....	....	68 500	29 600	....	39 800	....	....
3.........	51 800	....	69 200	27 200	....	38 900	....	....
Average.	52 800	....	68 700	28 800	....	39 200	183	175
30-Foot Struts.								
1.........	39 800	41 600	44 400	....	....	29 600	....	....
2.........	42 500	....	47 200	16 600	22 200	32 400	....	....
3.........	....	....	42 500	17 000	20 400	29 600	....	....
Average.	41 200	....	44 700	16 800	21 300	30 500	245	147

The reading of the pointer was discontinued after a marked permanent set was shown, or when the temporary shortening was so much as to exceed the limits of the contrivance; in three instances it was possible to watch the pointer up to the failure of the strut.

The permanent set in two tests was not obtainable, one in the short strut of nickel steel, as already explained, and the other in the first long strut tested—Strut No. 1 of carbon steel. The movement of this strut disturbed the extensometer so that the readings taken are in doubt.

The short struts failed by the buckling of the angles and side plates, in four cases at the middle panel, and in two cases at the panel next to the middle. The first carbon-steel strut crumpled upward, the others sideways.

The long struts failed by bending. The distortion of the pin-hole was not appreciable.

The amount of deflection at the middle of the long strut after failure varied from 0.64 to 2.60 in.

These tests were conducted with great care, and the figures given are, it is believed, correct. They are based on data furnished by the Phœnix Iron Company as to the relation existing between the mercury column and the pressure on the piston of the machine.

An examination of Table 27 shows that there is no evidence of an elastic limit. The total permanent set was exceedingly small, especially in the nickel-steel struts. The readings, however, are very uniform.

The compression in the 10-ft. struts, under equal loads, seems to be about the same for both steels, up to the point where the carbon steel shows a permanent set; and it then becomes greater for the carbon steel. The amount of bending of the long struts is the same for both steels up to the point of failure of the carbon steel.

The results are shown in detail in Tables 51 and 52 and in the photographs, Figs. 1 and 3, Plate XVIII. The nickel steel in this test compares very favorably indeed with the carbon steel, the short struts being three-quarters and the long struts one-half as strong again as those of carbon steel.

These tests were made in June, 1906.

Coefficient of Elasticity.

The coefficient or modulus of elasticity is the ratio of the unit-stress to the unit-deformation, within the elastic limit of the material. As its value varies inversely with the deformation, it may be regarded as a measure of the stiffness of the material. The less the change in length under a given stress the greater is the coefficient of elasticity and the stiffer is the material.*

Six specimens were prepared, three of nickel steel and three of carbon steel, as follows:

No. 158 from the 12 by 1-in. plate of nickel steel, Heat No. 17 673.

B 1 from a 6 by 1-in. unannealed eye-bar of nickel steel, Heat No. 17 749.

B 1 from a 6 by 1-in. annealed eye-bar of nickel steel, Heat No. 17 749.

No. *C* 196 from a 12 by 1-in. plate of carbon steel, Heat No. 33 342.

No. *C* 162 from a 12 by 1-in. plate of carbon steel, Heat No. 33 342.

No. *C* 109 from a 12 by ¾-in. plate of carbon steel, Heat No. 33 342.

The specimens were turned for a distance of 9½ in. between fillets to a diameter as large as possible. This form was used because the construction of the Olsen extensometer made a round specimen desirable, and because the area of a turned round may be determined with greater accuracy than that of a rectangular section.

The loading was applied very slowly, and all readings were taken with the beam balancing. Within the elastic limit, a given load may be sustained for an indefinite period, but, as the piece begins to yield, the stretching relieves a part of the load. Restoring it again increases

*Merriman.

the stretch, and this condition continues until an equilibrium is established, when the beam will remain "balancing."

The extensometer read directly to $\frac{1}{1\,000}$ in. and by means of a vernier to $\frac{1}{10\,000}$ in. The first reading was always taken with a given load on the specimen—4 000 lb. for the larger pieces, and 2 000 lb. for the carbon-steel pieces. In one of the tables filed for reference in the Library of the Society, however, all readings have been reduced to an assumed zero stretch with no load on the specimen. The values obtained are not scientifically exact. They are as nearly correct as the apparatus and careful observations could make them. In several instances two sets of values for the coefficient are recorded; the "*a*" results are more nearly in agreement and it is believed are the more accurate. With pieces, "*B* 1," the range over which the "*b*" readings were taken is small, and, though a change seems to take place at 12 000 lb., it is only apparent, and, had intermediate readings been taken, it is probable that a series would have been obtained similar to those between 12 000 and 25 000 lb. In the case of the carbon steel, after the "*b*" readings had been taken, the load was removed and a second series taken as a check on the first, because of suspected errors—through slipping of either the apparatus or the grips. The range for the carbon steel was necessarily small, and it is believed that a more delicate apparatus should be used for such material.

The values for the two steels are:

For the nickel steel..	$E = 30\,075\,000$,	1-in.	plate, No. 158.
	$= 30\,440\,000$,	6 "	*E. B.*, *B* 1, Original.
	$= 30\,420\,000$,	6 "	*E. B.*, *B* 1, Annealed.
For the carbon steel..	$E = 28\,940\,000$,	1 "	plate, *C* 196.
	$= 29\,480\,000$,	1 "	plate, *C* 162.
	$= 29\,840\,000$,	$\frac{3}{4}$ "	plate, *C* 109.

The coefficient for the nickel steel is slightly higher, but the difference is not great; and, as probable errors in measurement of specimens or loads may be as high as 500 000, it may be assumed that below the elastic limit the two steels behave almost identically. Beyond the elastic limit, however, the yielding of the nickel steel is more gradual, and is smaller in amount.

Had the entire range, up to the yield point, been taken in each case, the nickel steel would still be found to have the higher coefficient, the difference being greater than before.

It will be noticed that the elastic limit, as determined by this method, is considerably below the yield point. Curves plotted from these readings show clearly the relationship between the two. After the amount of stretch up to the yield point had been noted, the tests were continued and the specimens broken with the results shown in Table 28:

TABLE 28.—RESULTS OF TESTS FOR COEFFICIENT OF ELASTICITY.

Mark.	Area, in square inches.	Load per Square Inch.			Elastic Ratio.		Elongation, in 8 in., per cent.	Reduction of area, per cent.	Fracture.
		Elastic limit, in pounds.	Yield point, in pounds.	Ultimate strength, in pounds.	Elastic limit, per cent.	Yield point, per cent.			
Nickel Steel.									
158....	0.760	36 800	56 600	104 600	35.2	54.1	17.3	47.9	S Cup.
B 1....	0.722	40 200	62 300	104 300	38.5	59.7	18.8	53.3	S ½ Cup Unannealed.
B 1....	0.727	35 800	56 400	102 800	34.8	54.9	18.8	47.0	S ½ Cup Annealed.
Carbon Steel.									
C 196..	0.754	15 900	22 500	58 100	27.4	38.7	30.8	58.6	S ½ Cup.
C 162..	0.590	11 900	25 400	59 000	20.2	43.1	32.5	59.7	S ¾ Cup.
C 109..	0.383	15 700	28 700	65 300	24.0	44.0	28.0	60.3	S Cup.

Comparing the results in Table 28 with those obtained in the tensile tests of A. A. S. M. specimens, it is seen that there is a close agreement in the case of the nickel steel, Table 56; in the case of the carbon steel, the yield point, as determined above, is lower than that obtained from the A. A. S. M. specimens, and this could have been anticipated, as the speed of the machine was different. The method just described allowed the full deformation of the piece to take place before the given load was increased; while, in the previous tests, the increase in load was continuous.

The tests of turned rounds also gave lower values for the yield point and ultimate strength, because of the removal of the rolled surface or skin.

From these tests it is also seen how, under certain conditions, the ratio of the elastic limit or the yield point to the ultimate strength becomes, for the nickel steel, 50% higher than for the carbon steel.

The very low elastic ratio reported in Table 31 for the 1-in. plates of carbon steel may be partly accounted for by the speed of the machine, having been 1 in. in 20 min., for these pieces, and 1 in. in 6 min. for the other pieces. The tests made at the plant of the Luken's Iron and Steel Company, Table 33, show, however, that not all the drop may be accounted for in this way.

It would be interesting to experiment further in this direction, and to obtain the amount of permanent set under given increments of load. Only two attempts were made to obtain data of this kind.

The load was removed from piece No. 158 after 30 000 lb. (39 500 lb. per sq. in.) had been reached and a set of 0.0001 in. was noted. The removal of load from *B* 1 (original bar) after 40 000 lb. (55 400 lb. per sq. in.) gave a set of 0.0018 in.

Further attempts were abandoned because of the danger of serious derangement of the extensometer and the interruption to the continuity of the readings.

Tests of Eye-Bar Material.

This material was rolled by the Carnegie Steel Company from the two 51-ton heats, Nos. 17 673 and 17 749, known as "ideal shape nickel steel."

The following sizes were tested:

No.	Size.	Heat No.	Size of ingot.	Weight of ingot.	Size of slab.	Weight of slab.
1	16 by 2 in.	17 673	25 by 30 in.	12 080 lb.	18¼ by 14¼ in.	3 580 lb.
3	8 by 2 in.	17 749	22 by 25 in.	8 200 lb.	10 by 8 in.	1 900 lb.
4	6 by 1 in.	17 749	18 by 20 in.	6 060 lb.	10 by 8 in.	1 345 lb.

The bars were forged and tested at the Ambridge Plant of the American Bridge Company.

Forging of 6-in. Eye-Bars.—On June 14th, 1906, the 6-in. bars were taken into the shop. The eighteen months' exposure had rusted the surface considerably.

The general method followed in forging these bars differed only from the usual practice in that the desired temperature was obtained more slowly. Along with the four nickel-steel bars, sixteen plain carbon-steel bars were heated and forged.

Each bar was stamped with a distinguishing mark—*B* 1, *B* 2, *B* 3 and *B* 4—for identification. From one end of each, two pieces, 18 in. long, were sheared for specimen tests, one piece being tested as rolled and the other after having been annealed with the forged bars.

The furnace used for heating was fired with gas. The bars were run in at 11.08 A. M. in couples, *B* 2 on *B* 3 and *B* 4 on *B* 1, the carbon-steel bars being put in at the same time.

At intervals, the temperatures of furnace and bars were obtained by the Le Chatelier electrical pyrometer, and, when this failed, by a Queen and Company optical pyrometer.

First Head.—The temperature of the furnace at 10.30 A. M. was 1 200° (650° cent.), at 11.20, twelve minutes after the bars had been entered, it was 1 115° (600° cent.). The temperature at the time of forging was not obtained, as, through a misunderstanding, the heater removed the bars before the stated time. After the rough heads (known as cobbles) had been formed on the pile of two bars, they were then rolled, still together, and then brought back to the furnace. Practically all the upsetting was done at this operation, the second upsetting simply perfecting the shape of head.

The second heating was carefully watched; the "cobbles" were entered about 12.15 P. M.; at 12.40 P. M. the temperature of the furnace was 1 960° (1 070° cent.); at 12.52 P. M., it was 1 975° (1 080° cent.);

and after this reading the thermo-junction of the pyrometer separated. At 1.20 P. M. the temperature was 2 000° (1 090° cent.); at 1.30 P. M. 2 100° (1 150° cent.); and at 1.35 P. M., 2 100° (1 150° cent.). Bars *B* 4 and *B* 2 were then removed for final upsetting. The furnace was then forced to a higher temperature, reaching 2 200° (1 200° cent.) at 1.42 P. M., when Bar *B* 1 was removed. At 1.45 P. M., Bar *B* 3 was removed, the temperature being about 2 250° (1 230° cent.). The head of Bar *B* 1 was dished in punching, which necessitated reheating and flattening under a hydraulic ram.

After the final upsetting, the bars were rolled singly and then punched, and the ears were sheared off, after which each bar was again rolled and laid on edge on skids until the entire heat was forged.

The scale formed during the heating was removed by rapid hammering both before and after upsetting.

Annealing of 6-in. Eye-Bars.—In order, if possible, to obtain a variation in the heat treatment of the bars, they were run into the furnace in pairs, with the short lengths for specimen tests on top, as shown in Fig. 74.

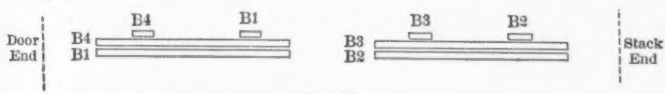


FIG. 74.

Bars *B* 1 and *B* 2 rested on the rolls and were separated from Bars *B* 4 and *B* 3 by ½-in. square rods. The specimen pieces were placed about 4 ft. from the ends of the bars.

The free circulation of the gases through the pin-holes and the possible overheating of the head were prevented by placing caps, 4 in. square, over the holes.

All temperature readings were taken with the electrical pyrometer; this was calibrated before being used. The gun of the pyrometer was inserted through small rectangular openings, about 4½ in. square, 6 ft. apart and 15 in. above the tops of the rolls and 13 in. above the eye-bars; the temperature of the furnace recorded, therefore, is that about 1 ft. above the top bar. The place occupied by the bars was probably from 50 to 100° (10 to 40° cent.) cooler than that indicated.

The bars were rolled into the furnace at 9.40 A. M., June 16th. The furnace had been operated continuously during the night, and was, therefore, thoroughly heated.

The heating of Bars *B* 4 and *B* 1 occupied 3 hr. 23 min., and of Bars *B* 3 and *B* 2, 3 hr. 50 min.

The probable average temperatures of the bars, when withdrawn, were:

B 1..................	1 500° (820° cent.)
B 2..................	1 450° (790° ")
B 3..................	1 500° (820° ")
B 4..................	1 550° (840° ")

Forging of 8 by 2-in. Eye-Bars.—The same marks that were stamped on the 6-in. bars were put on the 8 by 2-in. bars for identification. These bars were heated in an oil-fired furnace somewhat in need of repairs, so that the heating could not be controlled accurately; the extreme ends were much the hotter. The bars were heated and forged by themselves.

In the first head:

Bar *B* 1 was in the furnace 74 min.,
" *B* 2 " " " " 76 "
" *B* 3 " " " " 83 "

The average length of each interval required for upsetting, rolling, etc., was 6 min., or a total of 18 min. for each bar.

Reheating was made necessary because of the great width of head relative to the width of bars, the excess being about 80 per cent.

On account of the poor condition of the furnace, the bars were shoved in about 3 in. further than usual.

In the second head:

Bar *B* 1 was in the furnace 58 min.,
" *B* 2 " " " " 59 "
" *B* 3 " " " " 77 "

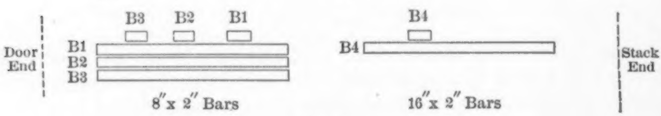


FIG. 75.

The average length of each interval required for upsetting, rolling, etc., of Bars *B* 1 and *B* 2 was 4 min., or a total of 12 min. for each bar. Bar *B* 3 was heated a fourth time, and the total time for upsetting, etc., was about 21 min.

The total time occupied with both heads was 3 hr. 27 min.

Forging of 16 by 2-in. Eye-Bar.—The 16 by 2-in. bar was heated and forged with five plain carbon-steel bars. The heating was done in the furnace used for the 8 by 2-in. bars, and the upsetting, etc., were done by the same men and machines. For identification it was stamped *B* 4.

The first head was in the furnace 1 hr. 45 min., the second, 91 min. The forging of the first head occupied 22 min., in three intervals; the second occupied 30 min., in five intervals.

The total time occupied was 4 hr. 10 min.

Annealing of 8 by 2-in. and 16 by 2-in. Bars.—These bars were annealed in the furnace already described. The heat treatment differed slightly, a higher temperature being attained.

At 10.25 P. M. the bars were rolled into the furnace in the order shown in Fig. 75.

The length of time the bars were in the furnace was 5 hr. 25 min.

At the time of withdrawal the temperatures of the heads of the bars nearest the door were, for Bar *B* 1, 1 500° (820° cent.); *B* 2, 1 440° (780° cent.), and *B* 3, 1 400° (760° cent.). The temperature of the 16-in. bar was not taken because of the breaking of the thermo-junction. Its temperature, however, was probably 1 600° (870° cent.), the stack end of the furnace being approximately 100° (40° cent.) higher than where the other bars were. The approximate average temperatures of the bars were probably as follows: *B* 1, 1 550° (840° cent.); *B* 2, 1 500° (810° cent.); *B* 3, 1 450° (790° cent.); *B* 4, 1 650° (900° cent.).

Tensile Tests of Full-Sized Eye-Bars.—These bars were tested in the new hydraulic testing machine of the American Bridge Company, at Ambridge, Pa. The pressure was supplied by a short-stroke, two-cylinder, Deeming hydraulic pump geared to an electric motor. The capacity of the machine is 4 000 000 lb., the pressure being read on a Shaw mercury gauge.

The only nickel-steel bars manufactured (within the writer's knowledge) up to the present time are those made by the American Bridge Company for the Blackwell's Island Bridge, New York City. With a knowledge of the difficulty in obtaining the yield point of nickel steel from the movement of the mercury column, it was agreed between the manufacturer and the engineer that an extensometer should be used for establishing this point, and that it should be assumed to be the load producing a permanent set of 0.025 in. in 20 ft. This method has not given entire satisfaction, because of the unusually low elastic limits reported at times.

It was intended, in this experimental work, to follow the same procedure; but the results thus obtained when considered in their relation to the readings as a whole and to the physical condition known as peeling are inconsistent. It is believed that in several instances the yield point had not been reached when the extensometer had recorded a set of 0.025 in. in 20 ft.

At the yield point there is always a marked change in the amount of stretch when the load is increased uniformly, and, between the elastic limit and this point, the stretch becomes gradually greater for each increment of load. Just before and at the yield point, the mill or furnace scale begins to fall from the bar because of its unelasticity. This is a certain indication that the bar is stretching rapidly.

In Tables 53 and 56, therefore, the results have been worked out on this basis, and not on an arbitrary assumption based on a certain permanent set.

This method for determining the elastic limit or yield point was the same as that used in the tests of struts, the load in increasing amounts being alternately applied and removed, and the stretch and

permanent set at each application noted by the extensometer. The readings are given in Tables 54 and 55, corrected, however, so that the zero readings for stretch and permanent set occur under no load.

Actually, the pointer was set in advance of the zero mark, as it was found impossible to set the extensometer pointer exactly at this mark. It was also found, in several instances, that, after a small load had been put on the bar, the pointer returned so far as to pass the original setting, and in the case of the 6-in. bar, *B* 3, it passed the limit of the graduated portion so that all the readings had to be estimated by the eye.

The several readings taken at two points within the elastic limit of the 8-in. and 16-in. bars, *B* 1 to *B* 4, Table 55, furnish some information as to the probable error of these results. It will be seen that differences of several thousandths occur. Such a series of readings for permanent set as occur for the 6-in. bar, *B* 1, where, between loads of 300 000 and 330 000 lb., there was no change, furnish another reason why too much importance should not be given to these results by themselves.

The complete graduated arc measured an extension of but 0.5 in., hence the readings were limited to this amount, which is far too small, it is thought, for obtaining all the information necessary in determining accurately the yield point. The readings for each bar were discontinued only when the limit of the extensometer had been reached.

Another objection to this procedure was the inability of the operator to sustain a constant pressure sufficiently long for a yielding of the material to take place, as described under "Coefficient of Elasticity" determinations. This was especially the case with the small bars; it was impossible to reduce the speed of the machine to what it should have been for experimental work. The speed could be varied within certain limits by an electric controller in connection with the motor operating the pump; and it also varied with each increase of load. Just before the maximum load for the 6-in. bars was reached the movement of head was about 3 in. in 1 min.; for the 8-in. bars, 1¼ in. in 1 min.; and for the 16-in. bars about ½ in. in 1 min. The 6 and 8-in. bars broke with a uniform silky fracture, and an average amount of elongation and reduction of area; and the drop of ultimate strength and elastic limit below those of the specimen unannealed tests was fairly uniform, so that the annealing described was evidently properly done. The number of tests is too small to draw any conclusions as to the effects or forging and annealing temperatures, the differences between the individual bars being very small.

The fine crystalline fracture of the 16-in. bar would give reason for thinking that it should have been held for a longer time in the annealing furnace and at a lower temperature. The fracture was

uniform, and gave no indication of maltreatment; but the elongations are greater than those of the smaller bars.

Coefficient of Elasticity—Full-Sized Bars.—It was desired to obtain the coefficient of elasticity in tension from these tests. For this purpose ten readings with the extensometer were taken of the stretch in each of the 8 and 16-in. bars, five with a given load as small as practicable, and five with a load just below the elastic limit (Tables 54 and 55). The readings vary slightly, as already noted, but, after excluding the doubtful ones, an average was used for the coefficient determination.

The following coefficients of elasticity were obtained:

8 by 2-in., $B\,1 - E = 26\,900\,000$
" " " " $B\,2 - E = 27\,910\,000$
" " " " $B\,3 - E = 27\,790\,000$
16 " 2 " $B\,4 - E = 26\,770\,000$

and, similarly, from the 6 by 1-in. bars, the approximate values:

6 by 1-in., $B\,1$ and $B\,4 - E = 27\,500\,000$
6 " " " $B\,2$ and $B\,3 - E = 25\,800\,000$

These values are much below the 30 000 000 obtained from the 1-in. round specimens cut from the 6-in. bar, *B* 1.

The most reasonable explanation of this difference is that the areas used in the calculations are too large. These were taken with calipers after all loose scale had been knocked off, and were the same as those used in obtaining the physical properties. The actual area of cross-section is always less than the section measured, because of the heavy scale resulting from rolling and annealing. Assuming the coefficient of elasticity to be 30 000 000, as obtained from the specimens, the extensometer readings would indicate that the loads per square inch given in Tables 53 and 56 are about 9% too small for the 6-in. bars and 7% and 6% too small for the 8-in. and 16-in. bars. Some of the errors may arise from the extensometer readings, but the fact that there is a close agreement among the bars of the same size would point to the first explanation. This scale is difficult to remove. It may be as much as $\frac{1}{8}$ in. thick on the heads; and a scale 0.04 in. thick would be sufficient to account for this difference.

Tensile Tests of Specimens of Eye-Bar Material.—The tensile tests of the specimens cut from the 6-in. bars, *B* 2, *B* 3 and *B* 4, were made by The Riehle Brothers Testing Machine Company of Philadelphia. The results are disappointing because of the failure of the Riehle autographic apparatus to work properly. The yield point had to be estimated from measurements obtained by a pair of dividers. The specimens from 8-in. and 16-in. bars, with the exception of *EB* 32, were tested at Drexel Institute on a 200 000-lb. Olsen machine

with an autographic apparatus. The two curves obtained for Specimens *EB* 11 and *EB* 21 are very different from the others at the yield point and, therefore, are marked abnormal. The variation in speed was not sufficient to affect the results materially.

The yield point, ultimate strength, and elastic ratio, Table 57, are lower for the annealed specimens in every instance but one (*EB* 32 broken by The Riehle Brothers Company). The elastic ratio of this piece is higher; but the yield point is in doubt, as no dividers were used. The elongation in 8 in. of the annealed specimens is less than in the unannealed in all but four instances. In two of these it is equal, and in the other two but 0.2% and 3.3% higher; and in the latter case, the fracture of the unannealed bar occurred near the end gauge mark, so that the stretch on one side was limited by the fillets. The reduction of area of annealed bars is lower in all but two instances—the two pieces cut from the 8-in. bar, *B* 3.

The differences are very small, and the large elongations and reductions of both the annealed and unannealed bars are indicative of a good quality of material.

The location of these specimen pieces in the eye-bar is a matter of interest. The center line of specimens cut from the 6-in. bars was located 1½ in. from the edge; from the 8-in. bars, two pieces were cut, one from the edge and one from the middle of the bar; and from the 16-in. bar only one 18-in. length was cut, and this was again sheared along the middle line, giving two pieces, 8 by 2 by 18 in., one of which was annealed with the bar. Pieces for bending tests were cut from the rolled edge, and the tensile pieces marked "2" next to these and 3 in. from the rolled edge. The piece marked "1" was cut from the sheared edge, so that *EB* 41 and *AEB* 41 were adjacent in the bars; the former being tested as rolled and the latter being annealed.

The yield point and tensile strength of the pieces cut from the edge and middle of the same bar are different. Edge pieces, with the exception of pieces *EB* 22 and *AEB* 22, gave lower results. These differences are practically eliminated by annealing.

TABLE 29.—COMPARISON OF RESULTS OF TESTS OF SPECIMENS.

Physical property.	Specimens Cut from 8-in. and 16-in. Eye-Bars.				Specimens Cut from 6-in. Eye-Bars.	
	Middle.		Edge.			
	Original.	Annealed.	Original.	Annealed.	Original.	Annealed.
Yield point.........	60 100	53 600	55 600	52 900	60 600	54 800
Ultimate strength..	101 500	97 800	98 500	96 600	103 100	100 000
Elastic ratio........	58.7%	54.9%	56.4%	54.8%	58.7%	54.5%
Elongation in 8-in...	21.1%	20.1%	21.9%	21.0%	19.4%	19.1%
Reduction of area..	48.5%	47.5%	49.7%	47.4%	51.1%	48.0%

Bending Tests of Eye-Bar Material.—As already described, pieces 18 in. long were cut from each bar before forging. One piece was annealed with the bar from which it was cut, but the other received no heat treatment whatever.

The specimens for bending were taken from the edge, and one side only was machined. They were 2 by 18 in. by the thickness of the bar.

The bending was done in the hydraulic bending machine at Pencoyd, Pa., in the American Bridge Company's shop.

A mandrel with end rounded to a radius of 2 in. was used for the specimens 2 in. thick, and the anvil was U-shaped with a mouth 8½ in. wide. The casting was broken in the first attempt. A similar, but heavier, anvil with a 9½-in. mouth was used throughout the series.

Each specimen was bent as much as possible by being forced into this opening by the 4-in. plunger, and then bent farther by pressure on the ends. It was desired that the radius of bend should be reduced to the thickness of the material; and in every instance the bending was greater than this.

The specimens from the 6-in. bars were similarly bent, the plunger being 2 in. instead of 4 in. in diameter, and the aperture of anvil being reduced to 5¾ in., thus increasing the amount of bending about the end of plunger. No definite amount of bending was contemplated, therefore some of the pieces were bent to cracking. Table 58 and the photograph of the bent specimens shown by Fig. 2, Plate XVII, give in detail the results of this bending.

The radius of the inner surface of the bends varied from 0.4 to 0.7 of the thickness of specimen, thus satisfying the usual requirements demanded of a medium carbon steel of 60 000-lb. ultimate strength.

APPENDIX A.

TABLE 30.—TENSILE TESTS ON A. A. S. M.
Made on 200 000-lb. Olsen

Test.	Original. Section, in inches.	Original. Area, in square inches.	Load per Square Inch. Yield point, in pounds.	Load per Square Inch. Ultimate strength, in pounds.	Elastic Ratio. Per cent.	Percentage of Elongation in: 2 in.	4 in.	6 in.	8 in.	Reduction of Area. Per cent.
12-in. Universal Plates.—Heat No. 17 673.										
Minimum	1.510 × 0.385	0.581	....	115 200	......	25.0	19.5	16.7	15.3	45.1
Average of 4.	1.510 × 0.385	0.581	....	116 300	......	29.3	20.8	18.0	15.8	46.4
Maximum ...	1.510 × 0.385	0.581	....	117 200	......	32.5	22.5	19.3	16.8	47.5
Minimum	1.505 × 0.385	0.579	66 700	112 800	58.8	21.0	19.5	16.7	13.8	43.9
Average of 4..	1.505 × 0.385	0.579	67 875	114 550	59.2	26.0	20.7	17.2	14.8	45.2
Maximum	1.505 × 0.385	0.579	70 100	116 600	60.1	29.0	21.5	17.7	15.6	45.9
Minimum	1.510 × 0.505	0.763	64 200	110 600	58.0	30.0	20.5	17.7	15.5	45.2
Average of 4..	1.510 × 0.505	0.763	64 200	113 020	58.0	31.0	22.7	18.5	16.4	46.2
Maximum	1.510 × 0.505	0.763	64 200	114 700	58.0	33.0	24.0	20.3	18.0	46.9
Minimum	1.500 × 0.505	0.758	61 400	106 700	57.3	26.0	23.5	19.0	16.6	49.1
Average of 3..	1.500 × 0.505	0.758	62 060	107 830	57.5	28.5	23.7	19.2	16.7	49.8
Maximum	1.500 × 0.505	0.758	62 700	108 400	57.8	30.0	24.0	19.5	16.8	50.7
Minimum	1.510 × 0.750	1.133	61 400	106 000	57.2	31.0	23.5	18.7	16.3	42.5
Average of 4..	1.510 × 0.750	1.133	61 830	106 900	57.9	33.5	25.0	20.3	17.5	46.6
Maximum	1.510 × 0.750	1.133	62 000	107 700	58.5	36.0	27.0	21.7	18.3	49.4
1 test only....	1.500 × 0.750	1.125	62 700	108 400	57.8	27.0	24.3	20.7	18.1	47.4
Minimum	1.250 × 1.005	1.256	55 400	106 800	51.6	37.0	26.0	21.3	19.3	45.8
Average of 4..	1.250 × 1.005	1.256	58 550	107 320	54.5	38.5	27.6	22.7	20.0	47.4
Maximum	1.250 × 1.005	1.256	61 600	107 900	57.4	40.0	30.5	24.7	21.5	48.2
Minimum	1.250 × 1.000	1.250	58 400	104 600	55.8	38.0	28.0	21.7	19.8	48.9
Average of 2..	1.250 × 1.000	1.250	58 500	104 600	55.9	39.0	28.7	22.0	19.8	49.1
Maximum	1.250 × 1.000	1.250	58 600	104 600	56.0	40.0	29.5	22.3	19.8	49.4
6 by 6 by $\frac{3}{4}$-in. Angles.—Heat No. 17 749.										
Minimum	1.510 × 0.750	1.133	60 700	102 000	59.5	30.0	24.0	20.0	18.0	40.8
Average of 4..	1.510 × 0.750	1.133	62 250	102 500	60.6	32.0	24.6	20.8	18.6	42.0
Maximum	1.510 × 0.750	1.133	65 600	103 700	63.3	34.0	25.0	22.0	19.0	42.7
8 by 8 by 1-in. Angles.—Heat No. 17 749.										
Minimum	1.250 × 1.019	1.273	53 300	97 100	54.6	28.0	27.5	22.7	20.0	46.0
Average of 4..	1.250 × 1.019	1.273	54 100	97 870	55.3	34.6	28.7	23.7	21.3	48.5
Maximum	1.250 × 1.019	1.273	55 000	98 800	56.6	41.0	30.5	25.3	22.8	51.1

NOTE.—This table is condensed from the original by giving, for each group of tests
* Distance from nearest end gauge mark. † Fracture slightly crystalline.

(PART II.)

Standard Specimens of Structural Steel.

Machine at Drexel Institute.

Fracture. Character.	Fracture. Location. Inches.*	Speed of breaking.	Location of specimen.	Remarks.
S. ½ Cup. S. ¾ Cup. S. 1 Cup.	2.2 2.9 3.3	1 in. in 6 min. to ultimate strength, then 1 in. in 2 min. to end.	Edge. “ “	Yield point lost. “ “ “ “ “ “
S. 1 Cup.† S. ¾ Cup. S. Ang. S. ¾ Cup.	1.2 2.2 .... 3.3	1 in. in 6 min. within yield point. 1 in. in 20 min. before and well beyond yield point, then 1 in. in 6 min. to end.	Interior Interior. Edge. Interior.	Yield point well marked.
S. ¾ Cup. S. 1 Cup. S. 1 Cup.	3.2 4.0 4.4	1 in. in 6 min. throughout. 1 in. in 6 min. well beyond yield point, then 1 in. in 2 min. to end.	Edge. “ “	Yield point fairly well marked. Yield point lost. “ “ “
S. ¾ Cup. S. ⅞ Cup. S. 1 Cup.=	2.8 3.8 4.3	1 in. in 6 min. within yield point. 1 in. in 20 min. before and well beyond yield point; 1 in. in 6 min. again; finally 1 in. in 2 min.	Interior. “ “	Yield point fairly well marked. “ “ “ “ “ “ “ “ “ “
S. ½ Cup. S. ½ Cup. S. ¾ Cup.	2.5 3.5 4.4	1 in. in 6 min, throughout. 1 in. in 6 min. within yield point; 1 in. in 20 min. before and well after yield point; then 1 in. in 6 min. to end.	Edge. “ “	Yield point lost. Yield point well marked. “ “ “ “
S. ½ Cup.	2.5	1 in. in 6 min., 1 in. in 20 min., 1 in. in 6 min.; finally 1 in. in 2 min. See No. 52.	Edge.	Yield point well marked.
S ½ Cup. S. ¾ Cup. S. 1 Cup.	3.5 3.8 4.7	1 in. in 6 min. within yield point. 1 in. in 20 min. before and well beyond yield point; then 1 in. in 6 min.; finally 1 in. in 2 min.	Edge. “ “	Yield point well marked. “ “ “ “ “ poorly marked.
S. ½ Cup. S. ¾ Cup. S. 1 Cup.	4.1 4.3 4.5	1 in. in 6 min. within yield point. 1 in. in 20 min. before and well beyond yield point; then 1 in. in 6 min. to end.	Interior. “ “	Yield point well marked. “ “ “ “ “ “ “ “
S. = Cup. S. = Cup. S. = Cup.	1.5 2.3 3.5	1 in. in 20 min. beyond yield point. 1 in. in 6 min. to end. 1 in. in 6 min. within yield point; 1 in. in 20 min. before and well beyond yield point; then 1 in. in 6 min. to end. 1 in. in 6 min. within yield point. 1 in. in 20 min. before and well beyond yield point; then 1 in. in 6 min. to end.	Edge. “ “	Yield point fairly well marked. “ “ “ “ “ “ “ “ “ “
S. ½ Cup. S. ¾ Cup. S. 1 Cup.	3.6 3.9 4.3	1 in. in 6 min. within yield point. 1 in. in 20 min. before and well beyond yield point; then 1 in. in 6 min.; finally 1 in. in 2 min. to end.	Edge. “ “	Yield point fairly well marked. “ “ “ “ “ “ “ “ “ “

minimum, average and maximum records, as shown in the first column.

TABLE 31.—TENSILE TESTS ON A. A. S. M.
Made on 200 000-lb. Olsen
All Specimens Lo-

Test.	Original.		Load per Square Inch.			Elastic Ratio.	
	Section, in inches.	Area, in square inches.	Yield point, in pounds.	Drop of beam, in pounds.	Ultimate strength, in pounds.	Yield point, per cent.	Drop of beam, per cent.
12-in. Universal Plates.—Heat No. 33 342.							
Minimum.......	1.510 × 0.355	0.536	37 300	39 700	62 700	51.9	61.7
Average of 4.....	1.510 × 0.355	0.536	38 800	40 800	63 720	60.5	64.1
Maximum........	1.510 × 0.355	0.536	41 400	42 500	64 400	66.1	67.8
16-in. Universal Plates.—Heat No. 41 520.							
Minimum.........	1.510 × 0.380	0.574	34 500	35 900	66 200	52.1	54.2
Average of 2.....	1.510 × 0.380	0.574	35 350	36 250	66 750	52.9	54.3
Maximum........	1.510 × 0.380	0.574	36 200	36 600	67 300	53.8	54.4
12-in. Universal Plates.—Heat No. 33 342.							
Minimum	1.510 × 0.475	0.717	36 500	37 100	63 000	57.6	58.1
Average of 4	1.510 × 0.475	0.717	36 880	37 320	63 770	57.8	58.5
Maximum	1.510 × 0.475	0.717	37 000	37 700	64 200	57.9	58.9
Minimum	1.510 × 0.755	1.140	32 500	33 700	61 900	52.5	54.4
Average of 4.....	1.510 × 0.755	1.140	33 420	34 050	62 150	53.8	54.7
Maximum	1.510 × 0.755	1.140	34 200	34 400	62 500	54.9	55.2
Minimum	1.250 × 1.000	1.250	26 400	27 000	59 200	43.3	45.4
Average of 6......	1.250 × 1.000	1.250	27 380	29 230	60 850	44.9	47.9
Maximum........	1.250 × 1.000	1.250	28 200	31 500	62 400	46.4	50.5
Minimum	1.250 × 0.995	1.244	25 700	26 900	58 500	43.8	45.5
Average of 2......	1.250 × 0.995	1.244	25 700	26 950	58 600	43.8	45.8
Maximum........	1.250 × 0.995	1.244	25 700	27 000	58 700	43.9	46.2

NOTE.—This table is condensed from the original by giving, for each group of tests,
* Distance from nearest end gauge mark.

STANDARD SPECIMENS. STRUCTURAL CARBON STEEL.

Machine at Drexel Institute.

cated at Edge.

Percentage of Elongation in:				Reduction of area, per cent.	Fracture.		Speed of breaking.
2 in.	4 in.	6 in.	8 in.		Character.	Location, inches.*	
43.0	35.0	30.3	26.9	55.0	S. Ang.	4.2	1 in. in 6 min to ultimate strength, then 1 in. in 2 min. to end.
45.7	35.7	31.1	27.9	56.2	S. Ang.	4.6	
47.5	37.5	32.7	29.4	56.9	S. Ang. S. ½ Cup.	4.9	
48.0	35.0	30.7	27.0	52.4	S. Ang.	3.2	1 in. in 6 min. to ultimate strength, then 1 in. in 2 min. to end.
49.0	35.7	31.0	27.2	55.3	S. Ang.	3.5	
50.0	36.5	31.3	27.5	58.2	S. Ang.	3.9	
50.0	35.0	29.0	26.3	54.2	S. Ang.	1.3	1 in. in 6 min. throughout.
51.2	37.1	30.4	27.3	56.5	S. Ang.	2.7	1 " " 6 " "
53.0	38.5	32.0	28.8	58.4	S. Cup. S. ½ Cup.	3.8	1 " " 6 " " 1 " " 6 " "
52.0	41.0	34.3	31.3	57.1	S. Cup.	3.8	1 " " 6 " "
53.7	42.0	35.3	32.2	58.0	S. Cup.	4.3	1 " " 6 " "
57.0	43.0	36.3	33.0	58.7	S. Ang. S. Cup.	5.1	1 " " 6 " " 1 " " 6 " "
47.0	41.0	34.0	31.0	56.3	S. ½ Cup.	4.2	1 in. in 6 min. within yield point; 1 in. in 20 min. before and well beyond yield point; 1 in. in 6 min. again; finally 1 in. in 2 min.
54.9	42.5	36.0	32.4	57.7	S. ¾ Cup.	4.7	
60.0	44.0	38.3	34.0	59.9	S. 1 Cup.	5.0	
58.0	42.5	36.7	32.5	60.4	S. Aug.	4.3	1 in. in 6 min. within yield point; 1 in. in 20 min. before and well after yield point; finally 1 in. in 6 min.
60.0	43.0	37.0	33.1	60.9	S. ½ Cup.	4.4	
62.0	43.5	37.3	33.8	61.5	....	4.5	

minimum, average, and maximum records as shown in the first column.

TABLE 32.—TENSILE TESTS ON "PARALLEL-SIDED" SPECIMENS OF STRUCTURAL NICKEL STEEL.
Made on 200 000-lb. Olsen Machines at Pencoyd Iron Works and at Lukens Iron and Steel Company.

Test.		Original.		Load per Square Inch.		Elastic ratio, per cent.	Elongation in 8 in., per cent.	Reduction of area, per cent.	Fracture.		Speed of Breaking.		Remarks.
		Section, in inches.	Area, in sq. in.	Drop of beam, in pounds.	Ultimate strength, in pounds.				Character.	Location. Inches.*	At drop of beam, 1 in. in:	At break, 1 in. in:	
12-in. Universal Plates.—Heat No. 17 673.	Minimum......	1.500 × 0.384	0.575	67 500	112 600	58.1	15.00	45.2	S. Ang.	0.0	40 sec.	40 sec.	Yield point lost.
	Average of 4..	1.500 × 0.384	0.575	69 000	115 450	60.2	15.94	46.7	S. Ang. S. Cup.	2.6	3 min. 40 sec.	3 min. 40 sec.	
	Maximum....	1.500 × 0.384	0.575	72 000	117 600	62.4	16.75	48.9	S. Ang.	4.3	3 min.	10 sec.	
	Minimum......	1.494 × 0.502	0.751	60 400	107 300	55.8	15.00	46.5	S. Ang.	2.2	3 min.	3 min.	Yield point lost.
	Average of 4..	1.494 × 0.502	0.751	62 830	111 200	56.5	17.56	47.9	S. Ang. S. Ang.	3.5	40 sec. 40 sec.	40 sec. 40 sec.	
	Maximum.....	1.494 × 0.502	0.751	65 300	113 600	57.5	20.00	50.2	S. Ang.	4.5	3 min.	10 sec.	
	Minimum......	1.500 × 0.750	1.125	62 600	108 200	56.1	16.50	37.6	S. ½ Cup.	1.9	3 min.	3 min.	Yield point questionable.
	Average of 4..	1.500 × 0.750	1.125	66 150	110 370	59.1	17.56	41.0	S. Cup. S. ½ Cup.	3.2	40 sec. 3 min.	40 sec. 10 sec.	
	Maximum......	1.500 × 0.750	1.125	68 800	111 500	62.1	18.50	43.6	S. Ang.	4.0	40 sec.	40 sec.	
	Minimum......	1.242 × 1.000	1.242	58 400	103 400	56.5	20.00	46.4	S. ¾ Cup.	1.5	40 sec.	40 sec.	(a) Cap of Lukens machine not sufficient to break 1-in. piece in 10 sec.
	Average of 4..	1.242 × 1.000	1.242	61 320	104 600	58.6	20.87	49.1	S. ¾ Cup.	2.8	3 min. 40 sec.	3 min. 40 sec.	
	Maximum.....	1.242 × 1.000	1.242	63 700	106 500	60.9	22.00	50.6	S. 1 Cup.	4.4	3 min.	3 min.	
6 × 6 × ¾-in. Angles.—Heat No. 17 749.	Minimum......	1.500 × 0.757	1.136	64 900	103 000	62.7	16.75	28.2	S. Cup.	1.0	40 sec.	40 sec.	Slightly crystalline.
	Average of 4...	1.500 × 0.757	1.136	66 660	104 370	63.6	17.87	35.2	S. Ang. Square.	2.9	40 sec. 40 sec.	40 sec. 40 sec.	" " ¾ crystalline.
	Maximum.....	....	......	68 400	106 100	65.2	19.25	38.5	Irreg. Cup.	4.5	40 sec.	40 sec.	⅛ "
8 × 8 × 1-in Angles.—Heat No. 17 749.	Minimum.....	1.250 × 1.019	1.273	61 200	99 300	60.4	15.75	37.0	S. ½ Cup.	1.8	40 sec.	40 sec.	
	Average of 4...	1.250 × 1.019	1.273	62 070	100 300	61.8	18.50	41.0	S. ½ Cup. S. ¾ Cup.	2.6	40 sec. 40 sec.	40 sec. 40 sec.	
	Maximum.....	1.250 × 1.019	1.273	63 300	101 200	63.6	21.50	44.0	Irreg. Cup.	4.5	40 sec.	40 sec.	Badly pitted.

NOTE.—This table is condensed from the original by giving, for each group of tests, minimum, average, and maximum records.
*Distance from nearest end gauge mark.

TABLE 33.—TENSILE TESTS ON "PARALLEL-SIDED" SPECIMENS OF STRUCTURAL CARBON STEEL.

Made on 200 000-lb. Olsen Machines at Pencoyd Iron Works, Pencoyd, Pa., and at Lukens Iron and Steel Company, Coatesville, Pa.

Test.	ORIGINAL.		LOAD PER SQUARE INCH.		Elastic ratio. Per cent.	Percentage of elongation in 8 in.	Reduction of area. Per cent.	FRACTURE.		SPEED OF BREAKING.	
	Section, in inches.	Area, in square inches.	Drop of beam, in pounds.	Ultimate strength, in pounds.				Character.	Location. Inches.*	At drop of beam. 1 in. in:	At break. 1 in. in:
12-IN. UNIVERSAL PLATES.—HEAT NO. 33 342.											
Minimum	1.491 × 0.354	0.528	37 900	62 400	58.6	28.50	53.6	S. ½ Cup.	2.5	15 sec.	15 sec.
Average of 4	1.491 × 0.354	0.528	42 300	65 500	64.4	29.75	56.5	S. ¾ Cup. S. Cup.	4.4	3 min. 15 sec.	3 min. 15 sec.
Maximum	1.491 × 0.354	0.528	48 600	67 600	71.8	31.00	59.5	S. Ang.	5.5	3 min.	10 sec.
Minimum	1.516 × 0.482	0.730	33 600	62 000	51.4	26.25	52.5	S. Ang.	2.0	3 min.	3 min.
Average of 4	1.516 × 0.482	0.730	37 320	64 400	57.9	28.37	55.6	S. Cup. S. Cup.	3.0	15 sec. 15 sec.	15 sec. 15 sec.
Maximum	1.516 × 0.482	0.730	41 400	65 500	63.2	30.50	59.2	S. ¼ Cup.	5.3	3 min.	10 sec.
Minimum	1.491 × 0.756	1.128	30 500	61 100	48.7	32.00	53.6	S. ¾ Cup.	2.8	3 min.	3 min.
Average of 4	1.491 × 0.756	1.128	33 620	62 300	53.9	32.50	56.5	S. Cup. S Ang.	3.8	15 sec. 3 min.	15 sec. 10 sec.
Maximum	1.491 × 0.756	1.128	36 400	63 300	58.5	33.00	59.1	S. Ang.	4.5	15 sec.	15 sec.
Minimum	1.251 × 0.997	1.247	28 400	58 300	46.1	31.00	56.2	S. ½ Cup.	1.0	15 sec.	15 sec.
Average of 4	1.251 × 0.997	1.247	33 470	60 570	55.2	33.75	58.4	S. Ang. Irreg. Cup.	3.6	3 min. 15 sec.	10 sec. 15 sec.
Maximum	1.251 × 0.997	1.247	39 300	62 100	63.7	38.75	61.4	S. Cup.	5.0	3 min.	3 min.

NOTE.—This table is condensed from the original by giving, for each group of tests, minimum, average, and maximum records, as shown in the first column.

* Distance from nearest end gauge mark.

TABLE 34.—TENSILE TESTS—NICKEL STEEL—ON SPECIMENS 3 IN. WIDE, WITH HOLES PUNCHED $\frac{15}{16}$ IN. AND SUB-PUNCHED $\frac{11}{16}$ IN. AND REAMED TO $\frac{15}{16}$ IN. DIAMETER.
Made on 200 000-lb. Olsen Machine at Drexel Institute.

	Test.	Thickness, in inches.	Dimensions of Original Net Section. Total. Width, in inches.	Dimensions of Original Net Section. Total. Area, in square inches.	Dimensions of Fractured Section. Total. Width, in inches.	Dimensions of Fractured Section. Total. Area, in square inches.	Load per Square Inch. Yield point, in pounds.	Load per Square Inch. Ultimate strength, in pounds.	Elastic ratio. Per cent.	Percentage of Elongation in: 4 in.	Percentage of Elongation in: 8 in.	Reduction of area. Total. Per cent.	Character of fracture.
Heat No. 17 673.	Minimum......	3/8	2.05	0.789	1.89	0.630	60 500	113 800	52.4	5.0	3.0	13.4	S. Ang. S. 1/2 Cup.
	Average of 4...	3/8	2.06	0.794	1.91	0.660	66 550	114 700	58.0	5.6	3.3	16.9	Silky-Fine Crys.
	Maximum.....	3/8	2.08	0.801	1.96	0.694	75 500	115 500	66.0	6.5	3.5	20.5	S. Ang.
	Minimum......	3/8	2.05	0.789	1.97	0.715	66 000	99 600	62.1	2.5	1.5	7.1	Crystalline.
	Average of 4..	3/8	2.05	0.790	1.98	0.722	69 250	103 850	66.7	3.4	2.0	8.6	L. Crystalline. R. 1/2 S., 1/2 Crys.
	Maximum.....	3/8	2.06	0.793	1.99	0.734	72 000	106 300	68.5	4.5	2.5	9.4	Crystalline. L. Silky; R. Crys.
	Minimum.....	3/4	2.07	1.553	1.95	1.345	69 000	109 400	61.1	4.0	2.8	11.8	Crystalline.
	Average of 4..	3/4	2.07	1.557	1.95	1.356	70 620	111 550	63.3	4.6	3.0	12.8	Crystalline.
	Maximum.....	3/4	2.08	1.561	1.97	1.369	72 000	113 700	65.3	5.5	3.3	13.7	Crystalline.
	Minimum......	3/4	2.04	1.530	2.00	1.460	67 000	94 200	70.6	2.0	1.3	4.6	Crystalline.*
	Average of 4...	3/4	2.06	1.544	2.01	1.466	71 870	95 220	75.5	2.2	1.3	5.0	Crystalline.*
	Maximum.....	3/4	2.07	1.553	2.02	1.475	81 500	97 100	86.5	3.0	1.5	5.5	Crystalline.*
Heat No. 16 080.	Minimum......	3/8	2.05	0.783	1.90	0.652	66 400	95 300	68.0	7.0	3.8	16.4	S. 1/2 Cup.
	Average of 2...	3/8	2.05	0.786	1.91	0.656	66 900	96 450	69.3	7.2	4.1	16.5	S. Ang.
	Maximum.....	3/8	2.06	0.789	1.93	0.660	67 400	97 600	70.7	7.5	4.4	16.7	S. Ang
	Minimum......	3/4	2.05	1.497	1.94	1.319	64 900	92 700	63.6	3.5	1.8	7.2	Crystalline.
	Average of 2...	3/4	2.06	1.509	1.96	1.353	65 850	97 350	67.8	4.0	2.5	10.3	Crystalline.
	Maximum.....	3/4	2.07	1.522	1.98	1.388	66 800	102 000	72.1	4.5	3.3	13.4	

NOTE.—This table is condensed from the original by giving for each group of tests, minimum, average, and maximum records.
* These pieces were bent in punching and were straightened before breaking.

TABLE 35.—TENSILE TESTS—CARBON STEEL—ON SPECIMENS 3 IN. WIDE, WITH HOLES PUNCHED $\frac{15}{16}$ IN. AND SUB-PUNCHED $\frac{11}{16}$ IN. AND REAMED TO $\frac{15}{16}$ IN. DIAMETER.

Made on 200 000-lb. Olsen Machine at Drexel Institute.

Test.	Thickness, in inches.	Dimensions of Original Net Section. Total.		Dimensions of Fractured Section. Total.		Load per Square Inch.		Elastic ratio. Per cent.	Percentage of Elongation in:		Reduction of area. Total. Per cent.	Character of fracture.
		Width, in inches.	Area, in square inches.	Width, in inches.	Area, in square inches.	Yield point, in pounds.	Ultimate strength. in pounds.		4 in.	8 in.		
CARBON STEEL—HEAT NO. 33 342.												
Minimum	⅜	2.04	0.734	1.80	0.527	38 000	65 000	58.5	11.0	7.0	25.6	S. Ang.
Average of 4...	⅜	2.05	0.738	1.81	0.541	39 500	65 270	60.5	11.5	7.1	26.7	S. Ang. S. Ang.
Maximum	⅜	2.06	0.742	1.83	0.547	41 000	65 400	62.7	12.0	7.5	28.6	S. Cup.
Minimum	⅜	2.03	0.731	1.85	0.578	40 000	57 500	68.2	8.5	5.5	19.2	S. ⅛ Cup. S. ½ Cup.
Average of 4...	⅜	2.03	0.732	1.86	0.583	40 620	59 120	68.7	8.9	5.7	20.3	S. ½ Cup.
Maximum	⅜	2.04	0.734	1.88	0.591	41 000	60 100	69.6	9.5	6.0	21.3	S. Ang.
Minimum	¾	2.04	1.530	1.68	1.014	36 500	65 400	55.1	11.5	7.0	26.9	S. ¼-in. near hole. Fine Crystalline.
Average of 4 ..	¾	2.05	1.540	1.75	1.077	37 250	66 370	56.1	13.0	8.0	30.0	S. Ang.
Maximum	¾	2.07	1.553	1.82	1.119	37 500	68 100	57.3	14.0	8.5	34.1	S Ang. S. Ang.
Minimum	¾	2.05	1.538	1.96	1.394	41 000	57 000	70.3	3.5	2.5	6.9	Crystalline.*
Average of 4 ..	¾	2.05	1.540	1.97	1.411	41 500	58 420	71.0	5.0	3.3	8.4	" *
Maximum	¾	2.06	1.546	1.99	1.432	42 500	59 400	71.9	6.0	3.8	9.4	" *

NOTE.—This table is condensed from the original by giving, for each group of tests, minimum, average, and maximum records, as shown in the first column.

* These pieces were bent in punching, and were straightened before breaking.

TABLE 36.—TENSILE TESTS—NICKEL STEEL AND CARBON STEEL—ON SPECIMENS 3 IN. WIDE, WITH $\frac{15}{16}$-IN. HOLE, PUNCHED-RIVETED.

Made on 200 000-lb. Olsen Machine at Drexel Institute.

Test.	Thickness, in inches.	Dimensions of Original Net Section. Total. Width, in inches.	Dimensions of Original Net Section. Total. Area, in square inches.	Dimensions of Fractured Section. Total. Width, in inches.	Dimensions of Fractured Section. Total. Area, in square inches.	Load per Square Inch. Yield point, in pounds.	Load per Square Inch. Ultimate strength, in pounds.	Elastic ratio. Per cent.	Percentage of Elongation in: 4 in.	Percentage of Elongation in: 8 in.	Reduction of area. Total. Per cent.	Character of fracture.
Carbon Steel—Heat No. 33 342.												
Minimum......	3/8	2.04	0.715	1.91	0.585	41 000	59 200	69.4	9.5	5.3	16.0	S. Ang.
Average of 2...	3/8	2.04	0.716	1.91	0.594	41 250	59 300	69.6	10.2	5.8	17.1	} L. S. Ang.
Maximum.....	3/8	2.05	0.718	1.91	0.603	41 500	59 400	69.9	11.0	6.3	18.2	} R. S. Cup.
Minimum......	3/4	2.05	1.538	2.05	1.522	41 000	48 100	80.2	1.5	1.0	0.5	Coarse Crys.
Average of 4..	3/4	2.06	1.546	2.06	1.531	42 620	50 120	85.1	1.8	1.2	0.9	" " *
Maximum.....	3/4	2.07	1.553	2.07	1.542	44 000	51 100	88.4	2.0	1.5	1.6	" "
Nickel Steel—Heat No. 17 673.												
Minimum......	3/8	2.06	0.793	1.99	0.704	75 000	105 900	70.0	4.0	2.0	8.3	Fine Crys.
Average of 2...	3/8	2.06	0.793	1.99	0.715	76 500	106 550	71.9	5.0	2.5	9.7	S. Cup.
Maximum.....	3/8	2.06	0.793	2.00	0.727	78 000	107 200	73.8	6.0	3.0	11.2	....
Minimum......	3/4	2.05	1.538	2.03	1.496	74 000	86 300	80.3	1.5	0.7	2.3	Fine Crys.
Average of 4...	3/4	2.06	1.547	2.04	1.503	78 500	88 200	89.1	2.0	1.0	2.9	" "
Maximum......	3/4	2.07	1.553	2.05	1.508	82 500	92 300	95.7	3.0	1.5	3.7	" "

NOTE.—This table is condensed from the original by giving, for each group of tests, minimum, average, and maximum records as shown in the first column.

* These pieces were slightly bent in punching, and again in cutting off the rivet heads.

TABLE 37.—BENDING TESTS ON SPECIMENS OF STRUCTURAL NICKEL STEEL, 3 IN. WIDE, WITH HOLES PUNCHED $\frac{15}{16}$ IN. AND SUB-PUNCHED $\frac{11}{16}$ IN. AND REAMED TO $\frac{15}{16}$ IN. DIAMETER, WITH HOLES $\frac{15}{16}$ IN. DIAMETER, PUNCHED-RIVETED.

Thickness of material, in inches.	Character of hole.	Angle of bend, in degrees.	2 × (Radius of Bend): Inches.	2 × (Radius of Bend): "t"	Remarks.
12-in. Universal Plates—Heat No. 17 673.					
1/2	Reamed.†	77	1 3/8	2.8	Crack — one side — across — edges not filed.
"	" †	104 (121)	1*	2.0	" both sides — across — edges filed.
"	"	75	1 3/8	2.8	" one side — hole outward — edges not filed.
"	" †	108	13/16	1.6	" one side — across — edges filed.
3/4	"	100	1 1/8	1.5	" both sides — one across.
"	"	89 (96)	1 1/4*	1.7	" both sides — across and through.
1/2	Punched.†	57	1 15/16	3.9	" both sides — hole outward.
"	" †	57	2	4.0	" " " " "
"	" †	65	2 1/8	4.3	" " " one across.
"	"	58	1 3/4	3.5	" " " hole outward.
3/4	"	38 (46)	4*	5.3	" " " across and through.
"	"	39 (47)	3 7/8*	5.2	" " " " " "
1/2	Punched-riveted.†	54	2 1/8	4.3	" " " hole outward.
"	" †	33	4 11/16	9.4	" one " across and through.
"	" †	54	2 1/8	4.3	" both " hole outward.
3/4	"	29 (34)	5 3/4*	7.7	" " " across and through.
"	"	27 (34)	5 13/16*	7.7	" " " " " "

"t" = Thickness of piece.
* Angle and radius of bend were estimated to be those existing when crack first started in piece.
† These pieces had been stressed to 32 000 lb. per sq. in. before being bent.

TABLE 38.—BENDING TESTS ON SPECIMENS OF MEDIUM CARBON STEEL, 3 IN. WIDE, WITH HOLES PUNCHED $\frac{15}{16}$ IN. AND SUB-PUNCHED $\frac{11}{16}$ IN. AND REAMED TO $\frac{15}{16}$ IN. DIAMETER, WITH HOLES $\frac{15}{16}$ IN. DIAMETER, PUNCHED-RIVETED.

Thickness of material, in inches.	Character of hole.	Angle of bend, in degrees.	2× (Radius of Bend): Inches.	2× (Radius of Bend): "t"	Opening at 1 in. from inside edge, in inches.	Remarks.
12-in. Universal Plates—Heat No. 33 342.						
½	Reamed	190	7/16	0.9	11/16	Crack—both sides, hole—outward—edges not filed.
½	"	193	7/16	0.9	13/16	" " " " " " " "
½	"	190	7/16	0.9	5/8	" " " " " edges filed.
½	"	198	11/16	1.4	1 1/16	" " " " " edges not filed.
¾	"	198	13/16	1.1	1 3/16	" " " " "
¾	"	193	3/4	1.0	1 3/8	" " " " "
½	Punched	191	11/16	1.4	1 1/4	" " " " "
½	"	183	3/4	1.5	1 7/16	" " " " "
½	"	179	7/8	1.8	1 9/16	" " " " "
½	"	196	5/8	1.3	1 1/4	" " " " "
¾	"	86	1 1/2	2.0	......	" " " " "
¾	"	82	1 7/8	2.5	......	" " " " "
½	Punched-riveted	172	1	2.0	1 3/4	" " " " "
½	"	175	7/8	1.8	1 7/16	" " " " "
½	"	184	15/16	1.9	1 1/2	" " " " "
½	"	169	15/16	1.9	1 13/16	" " " " "
¾	"	60(69)*	2 5/8	3.5	......	" " " across and through.
¾	"	64	2 3/8	3.2	......	" " " hole outward.

"t" = Thickness of piece.

* Angle and radius of bend were estimated to be those existing when crack first started in piece.

TABLE 39.—DRIFTING TESTS: OF STRUCTURAL NICKEL STEEL AND CARBON STEEL, ACCORDING TO SPECIFICATIONS.

Mark.	Diameter.			Enlargement.		Distance, center of hole to edge, in inches.	Character of edge.	Remarks.
	Original, in inches.	Enlarged.		Specified. Per cent.	Actual. Per cent.			All holes punched full size.
		Specified, in inches.	Actual, in inches.					
NICKEL STEEL.—12 BY $\frac{3}{8}$-IN. UNIVERSAL PLATE.—HEAT NO. 17 673. MARK *DS* 20.								
A	13/16	1	1 1/32	25	27	1 5/8	Sheared.	No cracks—hole or edge—either side.
B	15/16	1 3/16	1 3/16	25	27	1 7/8	"	" " " " " " "
C	15/16	1 7/32	1 3/16	30	27	1 13/32	Rolled.	" " " " " " "
D	15/16	1 1/4	1 7/32	33	30	1 7/8	Sheared.	" " " " " " "
E	13/16	1 1/16	1 3/32	33	35	1 5/8	"	" " " " " " "
F	13/16	1 1/8	1 1/8	38	38	1 1/2	Rolled.	" " " " " " "
G	13/16	1 7/32	1 3/16	50	46	1 5/8	Sheared.	" " " " " " "
H	15/16	1 13/32	1 3/8	50	47	1 7/8	"	Fair-sized crack at hole, die side only.
I	15/16	1 13/32	1 5/16	50	40	1 1/2	"	Crack ½ in. long from edge of plate; no cracks at hole.
J	15/16	1 13/32	1 11/32	50	43	1 1/2	Rolled.	Four fair-sized cracks at hole, die side only.
K	13/16	1 1/2	1 15/32	85	81	1	"	Four large cracks at hole, punch side only.
L	13/16	1 1/2	1 7/32	85	50	1 1/4	Sheared.	Crack ½ in. long from edge of plate; no cracks at hole.

NOTE.—Two interior holes in nickel steel marked *C* were drifted to 1 3/16 in. in diameter, the inside hole had one slight crack.

Mark.	Original, in inches.	Enlarged, specified, in inches.	Enlarged, actual, in inches.	Enlargement specified. Per cent.	Enlargement actual. Per cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks.
CARBON STEEL.—12 BY $\frac{3}{8}$-IN. UNIVERSAL PLATE.—HEAT NO. 33 342. MARK *CDS* 20.								
A	13/16	1	1 1/16	25	31	1 5/8	Sheared.	No cracks, hole or edge—either side.
B	15/16	1 3/16	1 5/16	25	40	1 7/8	"	" " " " " " "
C	15/16	1 7/32	1 5/32	30	23	1 13/32	Rolled.	Four slight cracks at hole—die side only.
D	15/16	1 1/4	1 7/32	33	30	1 7/8	Sheared.	No cracks.
E	13/16	1 1/16	1 1/16	33	31	1 5/8	"	" "
F	13/16	1 1/8	1 1/4	38	54	1 1/2	Rolled.	" "
G	13/16	1 7/32	1 3/16	50	48	1 5/8	Sheared.	" "
H	15/16	1 13/32	1 13/32	50	50	1 7/8	"	Small crack at hole—die side only.
I	15/16	1 13/32	1 7/16	50	53	1 1/2	"	No crack.
J	15/16	1 13/32	1 13/32	50	50	1 1/2	Rolled.	" "
K	13/16	1 1/2	1 1/2	85	85	1	"	Small cracks at hole, die side only.
L	13/16	1 1/2	1 3/8	85	69	1 1/4	Sheared.	Small cracks at hole, die side only. Crack ¾ in. long from edge of plate.

NOTE.—Two interior holes in carbon steel marked *C* were drifted to 1 3/32 in. and 1 1/8 in. in diameter, respectively. Slight cracks, four in number, were found on one side where bulging was excessive.

TABLE 40.—DRIFTING TESTS: STRUCTURAL NICKEL STEEL AND CARBON STEEL, ACCORDING TO SPECIFICATIONS.

Mark.	Diameter. Original, in inches.	Diameter. Enlarged. Specified, in inches.	Diameter. Enlarged. Actual, in inches.	Enlargement. Specified. Per cent.	Enlargement. Actual. Per cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks. All holes punched full size.
Nickel Steel.—12 by ¾ in. Universal Plate.—Heat No. 17 673. Mark *DS* 113.								
A	13/16	1	1	25	25	1 5/8	Sheared.	No cracks, hole or edge, either side.
B	15/16	1 3/16	1 5/32	25	23	1 7/8	"	" " " " " " "
C	15/16	1 7/32	1 7/32	30	30	1 13/32	Rolled.	" " " " " " "
D	15/16	1 1/4	1 1/4	33	33	1 7/8	Sheared.	" " " " " " "
E	13/16	1 1/16	1 1/64	33	25	1 5/8	"	" " " " " " "
F	13/16	1 1/8	1 1/8	38	38	1 1/2	Rolled.	One fair-sized crack at hole, die side only.
G	13/16	1 7/32	1 3/16	50	46	1 5/8	Sheared.	No cracks.
H	15/16	1 13/32	1 3/8	50	47	1 7/8	"	One fair-sized crack at hole, punch side only.
I	15/16	1 13/32	1 7/32	50	30	1 1/2	"	Crack ⅛ in. long from edge of plate; no cracks at hole.
J	15/16	1 13/32	1 11/32	50	43	1 1/2	Rolled.	No cracks.
K	13/16	1 1/2	1 1/2	85	85	1	"	Three large cracks at hole, punch side only.
L	13/16	1 1/2	1 3/16	85	46	1 1/4	Sheared.	Crack ⅛ in. long from edge of plate. No cracks at hole.

NOTE.—Two interior holes marked *C* were drifted to 1 7/32 in. in diameter; the middle hole had several fine cracks.

Mark.	Original, in inches.	Specified, in inches.	Actual, in inches.	Specified. Per cent.	Actual. Per cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks.
Carbon Steel.—12 by ¾-in. Universal Plate.—Heat No. 33 342. Mark *CDS* 113.								
A	13/16	1	1	25	25	1 5/8	Sheared.	No cracks, hole or edge, either side.
B	15/16	1 3/16	1 9/32	25	37	1 7/8	"	" " " " " " "
C	15/16	1 7/32	1 7/32	30	30	1 13/32	Rolled.	" " " " " " "
D	15/16	1 1/4	1 1/4	33	33	1 7/8	Sheared.	" " " " " " "
E	13/16	1 1/16	1 3/32	33	35	1 5/8	"	" " " " " " "
F	13/16	1 1/8	1 1/8	38	38	1 1/2	Rolled.	Several small cracks at hole, die side only.
G	13/16	1 7/32	1 1/4	50	54	1 5/8	Sheared.	No cracks.
H	15/16	1 13/32	1 7/16	50	53	1 7/8	"	" "
I	15/16	1 13/32	1 5/16	50	40	1 1/2	"	Crack ⅛ in. long from edge of plate; no cracks at hole.
J	15/16	1 13/32	1 3/8	50	47	1 1/2	Rolled.	Several small cracks at hole, die side only.
K	13/16	1 1/2	1 7/16	85	77	1	"	Four large cracks at hole, die side only.
L	13/16	1 1/2	1 1/2	85	85	1 1/4	Sheared.	Small crack at edge in corner only; no cracks at hole.

NOTE.—Two interior holes marked *C* were drifted to 1 7/32 in. in diameter; each had numerous small cracks.

TABLE 41.—DRIFTING TESTS: STRUCTURAL NICKEL STEEL AND CARBON STEEL, ACCORDING TO SPECIFICATIONS.

Mark.	Diameter.			Enlargement.		Distance, center of hole to edge, in inches.	Character of edge.	Remarks.
	Original, in inches.	Enlarged.						All drilled from the solid.
		Specified, in inches.	Actual, in inches.	Specified. Per cent.	Actual. Per cent.			
NICKEL STEEL.—12 BY 1-IN. UNIVERSAL PLATE.—HEAT No. 17 673. MARK *DS* 164.								
A	13/16	1	1 1/64	25	25	1 5/8	Sheared.	No cracks, hole or edge, either side.
B	15/16	1 3/16	1 7/32	25	30	1 7/8	"	" " " " " " "
C	15/16	1 7/32	1 1/2	30	60	1 13/32	Rolled.	" " " " " " "
D	15/16	1 1/4	1 7/32	33	30	1 7/8	Sheared.	" " " " " " "
E	13/16	1 1/16	1 1/8	33	38	1 5/8	"	" " " " " " "
F	13/16	1 1/8	1 3/32	38	35	1 1/2	Rolled.	" " " " " " "
G	13/16	1 7/32	1 11/64	50	44	1 5/8	Sheared.	" " " " " " "
H	15/16	1 13/32	1 11/32	50	43	1 7/8	"	" " " " " " "
I	15/16	1 13/32	1 1/4	50	33	1 1/2	"	Break from edge to hole; no other cracks.
J	15/16	1 13/32	1 3/8	50	47	1 1/2	Rolled.	No cracks.
K	13/16	1 1/2	1 7/32	85	50*	1	"	" "
L	13/16	1 1/2	1 1/8	85	38	1 1/4	Sheared.	Break from edge to hole; no other cracks.
NOTE.—Two interior holes marked *C* were drifted to 1 7/32 in. in diameter; no cracks.								
CARBON STEEL.—12 BY 1-IN. UNIVERSAL PLATE.—HEAT No. 33 342. MARK *CDS* 164.								
A	13/16	1	1	25	25	1 5/8	Sheared.	No cracks, hole or edge, either side.
B	15/16	1 3/16	1 1/4	25	33	1 7/8	"	" " " " " " "
C	15/16	1 7/32	1 1/4	30	33	1 13/32	Rolled.	" " " " " " "
D	15/16	1 1/4	1 7/32	33	30	1 7/8	Sheared.	" " " " " " "
E	13/16	1 1/16	1 1/16	33	31	1 5/8	"	" " " " " " "
F	13/16	1 1/8	1 1/8	38	38	1 1/2	Rolled.	" " " " " " "
G	13/16	1 7/32	1 3/16	50	46	1 5/8	Sheared.	Slight crack at corner of edge—one side; no cracks at hole.
H	15/16	1 13/32	1 13/32	50	50	1 7/8	"	Crack ⅛ in. long from edge of plate; no cracks at hole.
I	15/16	1 13/32	1 5/16	50	40	1 1/2	"	Break from edge to hole; no other cracks.
J	15/16	1 13/32	1 11/32	50	43	1 1/2	Rolled.	No cracks.
K	13/16	1 1/2	1 9/16	85	93	1	"	Four large ones at hole, punch side; small ones, die side.
L	13/16	1 1/2	1 1/2	85	85	1 1/4	Sheared.	Crack ⅛ in. long from edge of plate; no cracks at hole.

NOTE.—Two interior holes marked *C* were drifted to 1 1/4 in. in diameter; no cracks.
*Through an oversight, this hole was not enlarged further.

TABLE 42.—DRIFTING TESTS: STRUCTURAL NICKEL STEEL AND CARBON STEEL—HOLES DRIFTED TO CRACKING.

Mark.	Character of hole.	Diameter. Original, in inches.	Diameter. Enlarged, in inches.	Enlargement. Per cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks.
NICKEL STEEL.—12 BY $\frac{3}{8}$-IN. UNIVERSAL PLATE.—HEAT NO. 17 673. MARK *DC* 21.							
M	Punched.	15/16	1 15/32	57	1 1/2	Sheared.	Break—edge to hole. Large crack at hole, punch side only.
N	Drilled.	15/16	1 1/4	33	1 1/2	"	Crack ¼ in. long from edge of plate; no cracks at hole.
O	Reamed.	15/16	1 1/2	60	1 7/8	Rolled.	Three cracks (one large) at hole, punch side only.
P	Punched.	15/16	1 7/16	53	....	Interior.	One large crack at hole, punch side; two large cracks, die side.
Q	"	15/16	1 3/8	47	1 7/8	Rolled.	One small crack at hole, die side only.
R	"	15/16	1 13/32	50	1 7/8	"	Four large cracks at hole, die side only.
S	Drilled.	15/16	1 7/16	53	....	Interior.	Several large cracks at hole, punch side; one crack, die side.
T	"	15/16	1 1/2	60	1 7/8	Rolled.	One large crack at hole, punch side; two cracks, die side.
U	Reamed.	15/16	1 3/8	47	1 7/8	Sheared.	Crack ½ in. long from edge of plate; no cracks at hole.
V	Punched.	15/16	1 13/32	50	1 7/8	"	Crack ¾ in. long from edge of plate; no cracks at hole.
CARBON STEEL.—12 BY $\frac{3}{8}$-IN. UNIVERSAL PLATE.—HEAT NO. 33 342. MARK *CDS* 21.							
M	Punched.	15/16	1 5/8	73	1 1/2	Sheared.	Crack 1 in. long from edge of plate; no cracks at hole.
N	Drilled.	15/16	1 9/16	67	1 1/2	"	Crack ¼ in. long from edge of plate; six cracks at hole, die side.
O	Reamed.	15/16	1 23/32	83	1 7/8	Rolled.	Several large cracks at hole, die side.
P	Punched.	15/16	1 25/32	90	....	Interior.	Several small cracks at hole, die side.
Q	"	15/16	1 31/32	110	1 7/8	Rolled.	Small cracks at hole, punch side; two large cracks, die side.
R	"	15/16	2 1/32	117	1 7/8	"	Badly cracked at hole, both sides.
S	Drilled.	15/16	1 13/16	93	....	Interior.	Badly cracked at hole, die side.
T	"	15/16	1 7/8	100	1 7/8	Rolled.	Badly cracked at hole, die side.
U	Reamed.	15/16	1 5/8	73	1 7/8	Sheared.	Crack ½ in. long from edge of plate; cracks at hole, each side.
V	Punched.	15/16	1 5/8	73	1 7/8	"	Crack ¾ in. long from edge of plate; small cracks at hole, punch side.

TABLE 43.—DRIFTING TESTS: STRUCTURAL NICKEL STEEL AND CARBON STEEL—HOLES DRIFTED TO CRACKING.

Mark.	Character of hole.	Diameter. Original, in inches.	Diameter. Enlarged, in inches.	Enlargement. Per Cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks.
NICKEL STEEL.—12 BY $\frac{3}{4}$-IN. UNIVERSAL PLATE.—HEAT NO. 17 673. MARK *DC* 114.							
M.... ...	Punched.	15/16	1 7/16	53	1 1/2	Sheared.	Crack ⅛ in. long from edge of plate; no cracks at hole.
N........	Drilled.	15/16	1 1/4	33	1 1/2	"	Break from edge to hole; no other cracks at hole.
O........	Reamed.	15/16	1 3/4	87	1 7/8	Rolled.	Two small cracks at hole, punch side; four large cracks, die side.
P........	Punched.	15/16	1 5/8	73	....	Interior.	One large crack at hole, each side.
Q........	"	15/16	1 11/16	80	1 7/8	Rolled.	Three small cracks at hole, each side.
R........	"	15/16	1 3/4	87	1 7/8	"	Three fair-sized cracks at hole, each side.
S........	Drilled.	15/16	1 5/8	73	....	Interior.	Four small cracks at hole, punch side; two large cracks, die side.
T........	"	15/16	1 5/8	73	1 7/8	Rolled.	Three small cracks at hole, punch side; one large crack, die side.
U........	Reamed.	15/16	1 5/8	73	1 7/8	Sheared.	Break from edge to hole; one other small crack at hole, each side.
V........	Punched.	15/16	1 1/2	60	1 7/8	"	Crack 1 in. long from edge to hole; other small cracks, each side.
CARBON STEEL.—12 BY $\frac{3}{4}$-IN. UNIVERSAL PLATE.—HEAT NO. 33 342. MARK *CDC* 114.							
M........	Punched.	15/16	1 11/16	80	1 1/2	Sheared.	Crack ⅛ in. long from edge to hole; one small crack at hole, die side.
N........	Drilled.	15/16	1 11/16	80	1 1/2	"	Crack ¼ in. long from edge to hole; no cracks at hole.
O........	Reamed.	15/16	1 3/4	87	1 7/8	Rolled.	Three cracks at hole, die side.
P........	Punched.	15/16	2 3/16	133	....	Interior.	Six large cracks at hole, punch side, three cracks, die side.
Q.... ...	"	15/16	2 3/16	133	1 7/8	Rolled.	Ten large cracks at hole, die side; two cracks, punch side.
R........	"	15/16	1 9/16	67	1 7/8	"	Four large cracks at hole, die side.
S........	Drilled.	15/16	1 13/16	93	....	Interior.	One large crack at hole, punch side.
T........	"	15/16	2	113	1 7/8	Rolled.	Four cracks at hole, punch side; two, die side.
U........	Reamed.	15/16	1 1/2	60	1 7/8	Sheared.	Crack ¼ in. long from edge to hole; no cracks at hole.
V........	Punched.	15/16	1 9/16	67	1 7/8	"	Crack ½ in. long from edge to hole; one small crack at hole, each side.

TABLE 44.—DRIFTING TESTS: STRUCTURAL NICKEL STEEL AND CARBON STEEL—HOLES DRIFTED TO CRACKING.

Mark.	Character of hole.	Diameter. Original, in inches.	Diameter. Enlarged, in inches.	Enlargement. Per cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks.
NICKEL STEEL.—12 BY 1-IN. UNIVERSAL PLATE.—HEAT NO. 17 673. MARK *DC* 165.							
M	Drilled.	15/16	1 5/16	40	1 1/2	Sheared.	Crack ¼ in. long from edge to hole; no cracks at hole.
N	"	15/16	1 11/32	43	1 1/2	"	Crack ¼ in. long from edge to hole; no cracks at hole.
O	"	15/16	1 5/8	73	1 7/8	Rolled.	Two large cracks at hole, punch side; three, die side.
P	"	15/16	1 5/8	73	....	Interior.	Fine cracks at hole, punch side; six large cracks, die side.
Q	"	15/16	1 13/16	93	1 7/8	Rolled.	Five cracks (one large) at hole, punch side; two, die side.
R	"	15/16	1 9/16	67	1 7/8	"	Two large cracks at hole, punch side; no cracks die side.
S	"	15/16	1 11/16	80	....	Interior.	Four small cracks at hole, punch side; large cracks, die side.
T	"	15/16	1 13/16	93	1 7/8	Rolled.	Several cracks (one large) at hole, punch side; one, die side.
U	"	15/16	1 3/16	53	1 7/8	Sheared.	Break from edge to hole, two other cracks at hole, punch side.
V	"	15/16	1 7/16	53	1 7/8	"	Crack ½ in. long from edge to hole; one crack at hole, die side.
CARBON STEEL.—12 BY 1-IN. UNIVERSAL PLATE. MARK *CDS* 165.							
M	Drilled.	15/16	1 7/8	100	1 1/2	Sheared.	Crack ¼ in. long from edge to hole; one crack at hole, die side.
N	"	15/16	1 9/16	67	1 1/2	"	Crack ¼ in long from edge to hole; no cracks at hole.
O	"	15/16	2	113	1 7/8	Rolled.	Four large cracks at hole, punch side; four small cracks, die side.
P	"	15/16	2	113	....	Interior.	Four large cracks at hole, die side; small crack, punch side.
Q	"	15/16	2	113	1 7/8	Rolled.	Seven large cracks at hole, die side.
R	"	15/16	2	113	1 7/8	"	Six large cracks at hole, die side; one crack, punch side.
S	"	15/16	2 1/8	127	....	Interior.	Several cracks at hole, each side.
T	"	15/16	2 1/16	120	1 7/8	Rolled.	Five large cracks at hole, die side; one small crack, punch side.
U	"	15/16	1 9/16	67	1 7/8	Sheared.	Crack ¼ in. long from edge to hole; fine cracks at hole, punch side.
V	"	15/16	1 11/16	80	1 7/8	"	Crack ½ in. long from edge to hole; no cracks at hole.

TABLE 45.—"BEARING-ON-PINS" TESTS: TO DETERMINE AMOUNT OF YIELDING OF PIN AND SUPPORTS.

Readings taken with Olsen's Compression Micrometer.

Load.	Micrometer Readings, in Inches.								
	Pin No. 1.				Pin No. 2.				
	No. 1.	No. 2.	No. 3.	No. 4.	No 5.	No. 6.	No. 7.	No. 8.	No. 9.
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
*5 000	0.0018	0.0014	0.0009	0.0011	0.0017	0.0014	0.0016	0.0008	0.0011
10 000	0.0035	0.0023	0.0019	0.0022	0.0031	0.0026	0.0029	0.0017	0.0028
*15 000	0.0047	0.0033	0.0027	0.0029	0.0037	0.0030	0.0034	0.0022	0.0035
20 000	0.0052	0.0040	0.0034	0.0036	0.0042	0.0033	0.0039	0.0025	0.0040
*25 000	0.0064	0.0047	0.0037	0.0042	0.0048	0.0039	0.0044	0.0026	0.0042
30 000	0.0072	0.0053	0.0040	0.0048	0.0054	0.0044	0.0049	0.0029	0.0048
*35 000	0.0077	0.0058	0.0045	0.0050	0.0059	0.0047	0.0051	0.0032	0.0052
40 000	0.0081	0.0063	0.0047	0.0053	0.0065	0.0050	0.0054	0.0035	0.0054
*45 000	0.0086	0.0068	0.0050	0.0054	0.0070	0.0052	0.0056	0.0038	0.0057
50 000	0.0089	0.0073	0.0053	0.0058	0.0074	0.0056	0.0060	0.0041	0.0061
*55 000	0.0092	0.0077	0.0056	0.0061	0.0076	0.0059	0.0061	0.0042	0.0063
60 000	0.0095	0.0081	0.0061	0.0064	0.0080	0.0062	0.0063	0.0045	0.0066
*65 000	0.0099	0.0084	0.0061	0.0065	0.0083	0.0066	0.0065	0.0047	0.0068
70 000	0.0103	0.0088	0.0066	0.0067	0.0087	0.0070	0.0068	0.0049	0.0071
*75 000	0.0106	0.0093	0.0068	0.0068	0.0097	0.0073	0.0070	0.0952	0.0075
80 000	0.0111	0.0096	0.0071	0.0071	0.0105	0.0075	0.0072	0.0053	0.0078
*85 000	0.0116	0.0101	0.0075	0.0074	0.0110	......	......	......	......
90 000	0.0121	0.0106	0.0079	0.0076	0.0118	......	......	......	......
*95 000	0.0125	0.0110	0.0083	0.0078	......	......	......	......	......
100 000	......	0.0117	0.0089	0.0081	......	......	......	......	......

NOTE.—This table is condensed from the original.
* Interpolated values in horizontal lines marked in this way.

TABLE 46.—"BEARING-ON-PINS" TESTS: STRUCTURAL PLATE MATERIAL.

Readings taken with Olsen's Compression Micrometer.

Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Load.	MICROMETER READINGS, IN INCHES.					
	Nickel Steel—Heat No. 17 673.			Carbon Steel—Heat No. 33 342.		
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
*5 000	0.0017	0.0010	0.0010	0.0011	0.0013	0.0017
10 000	0.0020	0.0018	0.0020	0.0025	0.0023	0.0028
*15 000	0.0041	0.0027	0.0031	0.0035	0.0033	0.0041
20 000	0.0050	0.0037	0.0040	0.0045	0.0042	0.0052
*25 000	0.0057	0.0045	0.0048	0.0055	0.0052	0.0063
30 000	0.0064	0.0052	0.0054	0.0067	0.0066	0.0076
*35 000	0.0072	0.0060	0.0060	0.0083	0.0086	0.0098
40 000	0.0082	0.0067	0.0067	0.0111	0.0118	0.0124
*45 000	0.0091	0.0077	0.0073	0.0143	0.0156	0.0158
50 000	0.0101	0.0087	0.0083	0.0193	0.0206	0.0206
*55 000	0.0112	0.0100	0.0095	0.0255	0.0271	0.0265
60 000	0.0122	0.0120	0.0108	0.0333	0.0359	0.0346
*65 000	0.0135	0.0129	0.0121	0.0432	0.0477	0.0450
70 000	0.0148	0.0142	0.0135	0.0551	0.0625	0.0563
*75 000	0.0163	0.0151	0.0148	0.0696	0.0803	0.0693
80 000	0.0182	0.0167	0.0167	0.0850	0.0997	0.0871
*85 000	0.0204	0.0188	0.0188	0.1031	0.1201	0.1074
90 000	0.0229	0.0212	0.0210	0.1212	0.1420	0.1290
*95 000	0.0256	0.0238	0.0236	......	......	......
100 000	0.0288	0.0263	0.0265	......	......	......

NOTE.—This table is condensed from the original.
* Interpolated values in horizontal lines marked in this way.

TABLE 47.—"BEARING-ON-PINS" TESTS: STRUCTURAL PLATE MATERIAL.
AMOUNT OF COMPRESSION IN TEST PIECES.

Corrected for Yielding of Pin and Supports.

Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Load.	Amount of Compression, in Inches.					
	Nickel Steel—Heat No. 17 673.			Carbon Steel—Heat No. 33 342.		
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
*5 000	0.0004	−0.0002	−0.0001	−0.0003	−0.0001	0.0002
10 000	0.0007	−0.0004	−0.0002	0.0002	0.0000	0.0005
*15 000	0.0009	−0.0004	0.0001	0.0002	0.0000	0.0008
*20 000	0.0011	−0.0001	0.0003	0.0005	0.0002	0.0012
*25 000	0.0011	0.0001	0.0006	0.0008	0.0005	0.0016
30 000	0.0013	0.0003	0.0007	0.0014	0.0013	0.0023
*35 000	0.0016	0.0007	0.0010	0.0025	0.0028	0.0040
40 000	0.0022	0.0010	0.0013	0.0048	0.0055	0.0061
*45 000	0.0026	0.0017	0.0017	0.0075	0.0068	0.0089
50 000	0.0032	0.0022	0.0022	0.0120	0.0133	0.0133
*55 000	0.0039	0.0032	0.0031	0.0178	0.0194	0.0188
60 000	0.0045	0.0048	0.0041	0.0252	0.0278	0.0265
*65 000	0.0055	0.0055	0.0051	0.0348	0.0393	0.0366
70 000	0.0065	0.0064	0.0062	0.0463	0.0537	0.0475
*75 000	0.0076	0.0070	0.0072	0.0603	0.0710	0.0600
80 000	0.0092	0.0083	0.0089	0.0754	0.0901	0.0775
*85 000	0.0109	0.0100	0.0106	0.0930	0.1100	0.0973
90 000	0.0130	0.0120	0.0125	0.1106	0.1314	0.1184
*95 000	0.0153	0.0142	0.0147	......	......	......
100 000	0.0179	0.0162	0.0172	......	......	......

NOTE.—This table is condensed from the original.
* Interpolated values in horizontal lines marked in this way.

TABLE 48.—"BEARING-ON-PINS" TESTS: PLATE RIVET MATERIAL.
Readings taken with Olsen's Compression Micrometer.
Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Load.	MICROMETER READINGS, IN INCHES.					
	Nickel steel—Heat No. 2 096.			Carbon steel—Heat No. 19 241.		
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
*5 000	0.0018	0.0011	0.0014	0.0017	0.0011	0.0015
10 000	0.0034	0.0021	0.0033	0.0033	0.0024	0.0029
*15 000	0.0044	0.0031	0.0045	0.0048	0.0037	0.0044
20 000	0.0053	0.0039	0.0054	0.0064	0.0053	0.0061
*25 000	0.0060	0.0046	0.0063	0.0079	0.0068	0.0079
30 000	0.0066	0.0054	0.0072	0.0115	0.0100	0.0108
*35 000	0.0075	0.0063	0.0082	0.0160	0.0143	0.0148
40 000	0.0083	0.0072	0.0096	0.0206	0.0190	0.0194
*45 000	0.0197	0.0079	0.0115	0.0266	0.0247	0.0238
50 000	0.0114	0.0092	0.0132	0.0340	0.0313	0.0290
*55 000	0.0135	0.0107	0.0154	0.0482	0.0431	0.0364
60 000	0.0158	0.0128	0.0177	0.0635	0.0568	0.0487
*65 000	0.0181	0.0150	0.0202	0.0800	0.0732	0.0612
70 000	0.0205	0.0179	0.0231	0.1003	0.0885	0.0785
*75 000	0.0234	0.0217	0.0264	......	......	......
80 000	0.0271	0.0259	0.0298	......	......	......
*85 000	0.0308	......	......	......	......	......
90 000	0.0359	......	......	......	......	......

NOTE.—This table is condensed from the original.
* Interpolated values in horizontal lines marked in this way.

TABLE 49.—"BEARING-ON-PINS" TESTS: PLATE RIVET MATERIAL.
AMOUNT OF COMPRESSION IN TEST PIECES.

Corrected for Yielding on Pin and Supports.
Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Load.	AMOUNT OF COMPRESSION, IN INCHES.					
	Nickel Steel — Heat No. 2 096.			Carbon Steel — Heat No. 19 241.		
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
* 5 000	0.0003	0.0000	0.0005	0.0002	—0.0002	0.0005
10 000	0.0006	0.0001	0.0013	0.0005	—0.0001	0.0005
*15 000	0.0011	0.0006	0.0019	0.0016	0.0009	0.0013
20 000	0.0017	0.0010	0.0024	0.0028	0.0029	0.0026
*25 000	0.0018	0.0015	0.0031	0.0037	0.0031	0.0042
30 000	0.0019	0.0019	0.0037	0.0068	0.0059	0.0067
*35 000	0.0025	0.0025	0.0044	0.0110	0.0098	0.0102
40 000	0.0031	0.0031	0.0054	0.0154	0.0143	0.0146
*45 000	0.0042	0.0035	0.0069	0.0211	0.0197	0.0187
50 000	0.0056	0.0045	0.0084	0.0282	0.0260	0.0236
*55 000	0.0075	0.0059	0.0104	0.0422	0.0377	0.0307
60 000	0.0095	0.0077	0.0125	0.0572	0.0511	0.0428
*65 000	0.0115	0.0097	0.0148	0.0734	0.0673	0.0550
70 000	0.0136	0.0124	0.0174	0.0934	0.0823	0.0721
*75 000	0.0162	0.0159	0.0204	......	......	......
80 000	0.0197	0.0199	0.0236	.....	......	......

NOTE.—This table is condensed from the original.
* Interpolated values on horizontal lines marked in this way.

TABLE 50.—"Bearing-on-Pins" Tests: Nickel Steel and Carbon Steel.

Made on 100 000-lb. Olsen Machine; Readings Taken by Olsen's Compression Micrometer, Reading Directly to Thousandths of an Inch. Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Material.	Thickness, in inches.	Elastic limit, in pounds per square inch.	Amount of Compression, in Inches, Under Load, per Square Inch of:					Dimensions of Pin-Hole:		Increased thickness of metal over pin-hole, in inches.
			Elastic limit.	55 000	75 000	95 000	115 000	Original, in inches.	Distorted, in inches.	
Plate Carbon, 33 342	0.750	43 300	0.0021	0.0056	0.0196	0.0488	0.0972	1.00 by 1.00	1.09 by 1.03	0.08
	0.750	40 000	0.0014	0.0062	0.0210	0.0578	0.1156	1.00 " 1.00	1.12 " 1.04	0.07
	0.750	38 300	0.0020	0.0067	0.0204	0.0500	0.1026	1.00 " 1.00	1.10 " 1.04	0.06
Plate nickel, 17 673	0.750	......	......	0.0023	0.0040	0.0067	0.0114	1.00 " 1.00	1.02 " 1.01	0.00
	0.750	69 300	0.0024	0.0012	0.0036	0.0064	0.0105	1.00 " 1.00	1.02 " 1.01	0.01
	0.750	66 700	0.0022	0.0014	0.0034	0.0064	0.0110	1.00 " 1.00	1.02 " 1.01	0.02
Rivet carbon, 19 241	0.765	32 700	6.0036	0.0184	0.0490	0.1070	......	1.02 " 1.01	1.10 " 1.03	0.05
	9.765	31 400	0.0027	0.0174	0.0450	0.0950	......	1.02 " 1.02	1.10 " 1.04	0.05
	0.765	31 400	0.0039	0.0168	0.0366	0.0855	......	1.02 " 1.02	1.09 " 1.03	0.05
Rivet nickel, 2 096	0.710	59 200	0.0034	0.0030	0.0069	0.0124	0.0214	1.02 " 1.01	1.04 " 1.01	0.01
	0.710	62 000	0.0034	0.0029	0.0033	0.0112	0.0214	1.02 " 1.02	1.03 " 1.02	0.01
	0.710	53 500	0.0048	0.0050	0.0098	0.0160	0.0250	1.02 " 1.02	1.04 " 1.03	0.01
Averages.										
Plate carbon	0.750	41 200	0.0018	0.0062	0.0203	0.0522	0.1051	1.00 by 1.00	1.10 by 1.04	0.07
Plate nickel	0.750	68 300	0.0023	0.0016	0.0037	0.0065	0.0109	1.00 " 1.00	1.02 " 1.01	0.01
Rivet carbon	0.765	31 800	0.0034	0.0175	0.0435	0.0960	......	1.02 " 1.02	1.10 " 1.03	0.05
Rivet nickel	0.710	58 200	0.0039	0.0036	0.0067	0.0132	0.0226	1.02 " 1.02	1.04 " 1.02	0.01
Percentage That Above Average Values for Nickel Steel Are to Carbon Steel.										
Plate material		166.	128.	26.	18.	12.	10.			
Rivet material		183.	115.	21.	15.	14.	...			

TABLE 51.—(*Continued.*)

44 400	0.120		0.135								
45 400	0.125										
46 300	0.128		0.143								
47 200	0.130		0.143			0.001					
48 100	0.135		0.150			0.003					
49 000			0.153			0.004					
49 900	0.138		0.155	0.001		0.004					
50 900	0.143			0.001							
51 800	0.145		0.160	0.001		0.005					
52 800	0.148			0.003							
53 700	0.155		0.170	0.005		0.008					
54 600	0.155			0.005							
55 500	0.163		0.175	0.008		0.013					
56 400	0.163			0.009							
57 300	0.168			0.010							
58 500	0.173			0.013							
60 500	0.180			0.015							
61 900	0.190			0.020							
68 500				Failed.	0.33 Failed.						
69 200						Failed.					
Order of testing	5	1	6				2	3	4		

TABLE 51.—COMPRESSION TESTS OF STRUTS, 10 FT. LONG: STRUCTURAL NICKEL STEEL AND MEDIUM CARBON STEEL.

Made on 2 160 000-lb. Hydraulic Testing Machine, Phœnix Iron Company.

Load, in pounds per square inch.	Nickel Steel.						Carbon Steel.					
	Temporary shortening, in inches.			Permanent set, in inches.			Temporary shortening, in inches.			Permanent set, in inches.		
	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.
11 000	0.023			0.000	0.000	0.000		0.020	0.008	0.000	0.000	0.000
12 000			0.030									
14 800							0.025					
18 500	0.045		0.048									
20 400								0.045				
22 200							0.050		0.045			
24 100								0.060	0.050			
25 900								0.068	0.060			
26 800									0.065			0.003
27 800	0.073								0.070			0.008
29 200									0.075			0.009
29 600							0.075	0.085		0.005	0.005	
30 500									0.083			0.010
31 500								0.095			0.015	
31 800									0.093			0.018
32 700									0.100			0.023
33 700									0.110			0.030
33 900								0.110			0.018	
34 200	0.090											
35 100	0.095						0.103			0.020		
35 700								0.140			0.050	
37 000	0.100						0.125			0.070		
38 000	0.100									Failed.		
38 900			0.113				0.150			0.095		Failed.
39 800											Failed.	
40 700	0.110											
41 600			0.120									
41 800	0.113											
42 500	0.116		0.125									
43 500	0.118		0.126									

TABLE 52.—(*Continued.*)

38 000	0.505	0.458	0.403	0.003	....	....	....	....	....	....	....	....
38 900	0.515	0.473	0.415	0.003	....	....	....	....	....	....	....	....
39 800	0.525	0.483	0.420	0.005	....	....	....	....	....	....	....	....
40 700	0.535	0.485	0.426	0.008	0.001	....	....	....	....	....	....	....
41 600	....	0.503	0.428	0.015	0.001	0.000	....	....	....	....	....	....
42 500	....	0.508	....	0.018	0.005	Failed.	....	....	....	....	....	....
43 500	....	....	....	0.018	0.005	....	....	....	....	....	....	....
44 400	....	....	....	Failed.	0.008	....	....	....	....	....	....	....
46 300	....	....	....	....	0.009	....	....	....	....	....	....	....
47 200	....	....	....	....	Failed. 0.010	....	....	....	....	....	....	....
Order of testing	4	3	2				1	6	5			

NOTES.—Nickel-steel strut No. 1—Camber at no load, 0.20 in. up.
" " 18 500 lb. per sq. in., 0.00 in.
" " failure, 1.50 in. down.
" " 2—No record.
" " 3—Camber at failure, 2.25 in. down.

Carbon-steel strut No. 1—Camber at failure, 2.25 in. down.
Compressometer was dislodged twice during the test by the bending upward of the strut; the readings, therefore, are omitted from the table.
" " 2—Camber at no load, 0.34 in. up.
" " 13 900 lb. per sq. in., 0.37 in. up.
" " failure, 2.60 in. up.
" " 3—Camber at no load, 0.14 in. up.
" " 22 200 lb. per sq. in., 0.18 in. up
" " failure, 0.64 in. down.

TABLE 52.—COMPRESSION TESTS OF STRUTS, 30 FT. LONG: STRUCTURAL NICKEL STEEL AND MEDIUM CARBON STEEL.

Made on 2 160 000-lb. Hydraulic Testing Machine, Phœnix Iron Co.

Load, in pounds per square inch.	Nickel Steel.						Carbon Steel.					
	Temporary shortening, in inches.			Permanent set, in inches.			Temporary shortening, in inches.			Permanent set, in inches.		
	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.
11 200	....	....	0.120	0.000	0.000	0.000	0.095	0.122	0.117	0.000	0.000	0.000
12 000	0.130	....	....	....	....	....	....	....	....	....	....	....
13 000	....	0.135	....	....	....	....	0.118	0.147	0.144	....	0.001	....
13 900	0.170	....	....	....	....	....	0.128	0.152	....	....	0.001	....
14 800	0.185	....	....	....	....	....	0.158	0.172	0.174	....	0.003	....
15 700	0.195	....	....	....	....	....	0.178	0.184	....	....	0.003	....
16 600	....	....	....	....	....	....	0.193	0.199	0.204	....	0.005	0.004
17 600	....	....	....	....	....	....	0.200	0.214	....	....	0.009	....
18 500	0.240	0.235	0.220	....	....	....	....	0.224	0.234	....	0.010	0.009
19 400	....	....	0.230	....	....	....	....	0.237	....	....	0.010	....
20 400	....	....	0.245	....	....	....	0.230	0.239	0.264	....	0.013	0.015
21 300	....	....	0.248	....	....	....	0.243	....	0.279	....	....	0.018
22 200	....	....	0.270	....	....	....	0.245	0.264	0.284	....	0.015	0.020
23 100	0.298	0.283	0.283	....	....	....	0.260	....	0.307	....	....	0.025
24 100	....	0.298	0.283	....	....	....	0.270	0.289	0.329	....	0.020	0.030
25 000	....	0.298	0.298	....	....	....	0.283	....	0.347	....	....	0.035
25 900	....	0.313	0.315	....	....	....	0.283	0.314	0.359	....	0.023	0.040
26 800	....	....	0 318	....	....	....	0.293	....	0.389	....	....	0.050
27 800	0.358	0.343	0.328	....	....	....	0.303	0.349	0.414	....	0.033	0.060
28 700	....	....	0.335	....	....	....	....	....	0.437	....	....	0.070
29 600	0.400	0.363	0.340	0.001	....	....	....	0.384	....	Failed.	0.041	Failed.
30 500	....	....	0.345	....	....	....	....	....	....	....	....	....
31 500	0.430	0.388	0.345	0.001	....	....	....	0.399	....	....	0.053	....
32 400	....	0.395	0.365	....	....	....	....	....	....	....	Failed.	....
33 300	0.435	0.415	0.373	0.001	....	....	....	....	....	....	....	....
34 200	0.445	0.433	0.375	0.001	....	....	....	....	....	....	....	....
35 100	0.455	0.430	0.383	0.001	....	....	....	....	....	....	....	....
36 100	0.470	0.433	0.398	0.001	....	....	....	....	....	....	....	....
37 000	0.490	0.448	0.400	0.001	....	....	....	....	....	....	....	....

TABLE 53.—TENSILE TESTS OF 6 BY 1-IN., 8 BY 2-IN., AND 16 BY 2-IN. EYE-BARS: NICKEL-STEEL EYE-BAR MATERIAL.

Size, in inches.	HEAD A—PIN-HOLE. Original Dimensions. Width, in inches.	HEAD A—PIN-HOLE. Original Dimensions. Diameter, in inches.	HEAD A—PIN-HOLE. Elongated. Diameter, in inches.	HEAD A—PIN-HOLE. Excess, per cent.	HEAD B—PIN-HOLE. Original Dimensions. Width, in inches.	HEAD B—PIN-HOLE. Original Dimensions. Diameter, in inches.	HEAD B—PIN-HOLE. Elongated. Diameter, in inches.	HEAD B—PIN-HOLE. Excess, per cent.	LENGTH, IN FEET, BACK TO BACK OF PIN-HOLES. Before fracture.	LENGTH, IN FEET, BACK TO BACK OF PIN-HOLES. After fracture.	Gauge length, in feet.	Original Dimensions. Area, in square inches.	DIMENSIONS FRACTURED. Section, in inches.	DIMENSIONS FRACTURED. Area, in square inches.	LOAD, IN POUNDS PER SQUARE INCH. Yield point.	LOAD, IN POUNDS PER SQUARE INCH. Ultimate strength.	Elastic ratio, per cent.	PERCENTAGE OF ELONGATION IN: 12 inches.	PERCENTAGE OF ELONGATION IN: 10 feet.	PERCENTAGE OF ELONGATION IN: 20 feet.	Reduction of area, per cent.	FRACTURE. Character.	FRACTURE. Head A. Location, in feet.
6 by 1.	14.52	5.54	6.26	56.7	14.50	5.56	6.30	53.0	22.76	25.11	20.0	5.96	4.78 by 0.74	3.54	58 700	99 000	59.3	24.2	12.5	11.0	40.6	S. Ang.	3.4
6 " 1.	14.50	5.52	6.20	53.7	14.67	5.55	6.14	56.0	22.78	24.81	20.0	5.96	5.01 " 0.74	3.71	55 400	101 500	54.6	20.3	11.3	9.5	37.8	S. Ang.	4.8
6 " 1.	14.59	5.56	6.06	60.6	14.52	5.56	6.30	54.9	22.73	24.78	20.0	5.96	5.15 " 0.69	3.55	55 400	103 200	53.7	21.2	10.7	9.6	40.4	S. ½ Cup.	4.6
6 " 1.	14.51	5.54	6.14	55.6	14.50	5.54	6.06	57.0	22.77	24.79	20.0	5.88	4.80 " 0.66	3.17	56 100	100 300	55.9	25.8	10.6	9.5	46.1	S. ½ Cup.	3.1
8 " 2.	19.66	7.02	7.80	60.8	19.72	7.02	7.84	61.5	22.29	24.45	19.0	16.04	6.69 " 1.51	10.10	48 300	88 900	54.3	27.2	12.0	19 ft. 10.5	37.0	*S. Irreg.	21.1
8 " 2.	19.56	7.02	8.00	57.5	19.72	7.02	7.70	61.1	22.34	24.66	19.0	16.24	6.88 " 1.62	11.14	52 300	90 800	57.6	23.8	12.0	11.3	31.4	S. Cup.	3.2
8 " 2.	19.68	7.02	7.82	60.6	19.59	7.02	7.82	58.6	22.22	24.76	19.0	16.08	6.50 " 1.44	9.36	54 400	90 800	59.9	26.8	14.2	12.5	41.8	S. Cup.	18.6
16 " 2.	37.34	14.02	16.10	51.1	37.38	14.02	16.50	50.0	20.45	22.81	14.0	32.40	14.30 " 1.76	25.17	49 400	96 900	51.0	25.4	14.5	14 ft. 13.6	22.3	Cryst.	10.9

* 15% fine crystalline.

TABLE 54.—TENSILE TESTS OF 6 BY 1-IN. EYE-BARS, UP TO THE YIELD POINT. EYE-BAR MATERIAL—NICKEL STEEL—STRETCH AND PERMANENT SET.

Load on bar, in pounds per square inch.	*B* 1, AREA = 5.96 IN.		*B* 2, AREA = 5.96 IN.		*B* 3, AREA = 5.96 IN.		Load on bar, in pounds per square inch	*B* 4, AREA = 5.88 IN.	
	Stretch.	Set.	Stretch.	Set.	Stretch.	Set.		Stretch.	Set.
......	0.000	0.000	0.000	0.000	0.000	0.000	......	0.000	0.000
8 400	0.047	0.000	0.063	0.000	.....	0.000	8 500	.....	0.000
16 800	0.100	0.000	0.144	0.000	.....	0.000	17 000	0.094	0.000
25 200	.....	.....	0.175	0.000	.....	.....	......	.....	.....
33 600	0.222	0.000	0.231	0.000	0.231	0.000	34 000	0.206	0.000
41 900	.....	.....	0.300	0.006	.....	.....	......	.....	.....
43 600	.....	.....	0.303	0.000	.....	.....	......	.....	.....
45 300	0.300	0.002	0.325	0.000	0.313	0.000	45 900	0.288	0.006
46 100	.....	.....	0.328	0.006	.....	.....	......	.....	.....
47 000	0.313	0.003	0.338	0.006	0.325	0.006	47 600	0.303	0.009
47 800	0.313	0.005	0.350	0.013	0.331	0.009	48 500	0.313	0.011
48 700	0.322	0.006	0.363	0.019	0.338	0.016	49 300	0.319	0.013
49 500	0 325	0.008	0.363	0.022	0.350	0.025	50 200	0.325	0.013
50 300	0.338	0.009	0.369	0.023	0.363	0.028	51 000	0.338	0.014
51 200	0 341	0.009	0.375	0.023	0.363	0.031	51 900	0 338	0.016
52 000	0.347	0.009	0 394	0.025	0.375	0.034	52 700	0.350	0.019
52 900	0.356	0.009	0.400	0.027	0.375	0.038	53 600	0.363	0.020
53 700	0.363	0.009	0.406	0 028	0.388	0.041	54 400	0.369	.....
54 500	0.369	0.009	0.406	0.031	0.394	0.044	55 300	0.381	0.023
55 400	0.375	0.009	0.419	0.034	.....	0.050	56 100	0.394	0.027
					(0.419)				
56 200	0.388	0.011	0.428	0.041	0.444	0.056	57 000	0.406	0.038
57 100	0.394	0.013	.....	.....	.....	.....	......	peeling	.....
57 900	0.406	0.016	.....	.....	.....	.....	......	.....	.....
58 700	0.413	0.019	0.450	0.056	0.450	0 081	59 500	.....	0.338
	peeling								
59 600	0.431	0.038	.....	.....	.....	.....	60 400	.....	.....
60 400	.....	0.063	.....	.....	.....	.....	......	.....	.....
67 100	.....	.....	.....	.....	.....	0.506	......	.....	.....
Ultimate strength	99 000		101 500		103 200			100 300	
Order of testing	4		1		2			3	

The stretch and set were measured in a length of 15 ft.; they are expressed in decimals of an inch.

The first readings for Bars *B* 3 and *B* 4 were discarded because the pointer of the extensometer did not assume a permanent zero until after a load of 100 000 lb. had been applied.

TABLE 55.—TENSILE TESTS OF 8 BY 2-IN. AND 16 BY 2-IN. EYE-BARS, UP TO THE YIELD POINT. EYE-BAR MATERIAL—NICKEL STEEL—STRETCH AND PERMANENT SET.

Load on bar, in pounds per square inch.	8 BY 2-IN. BAR. B 1, AREA = 16.04 IN.		Load on bar, in pounds per square inch.	8 BY 2-IN. BAR. B 2, AREA = 16.24 IN.		Load on bar, in pounds per square inch.	8 BY 2-IN. BAR. B 3, AREA = 16.08 IN.		Load on bar, in pounds per square inch.	16 BY 2-IN. BAR. B 4, AREA = 32.40 IN.	
	Stretch.	Set.		Stretch.	Set.		Stretch.	Set.		Stretch.	Set.
......	0.000	0.000	......	0.000	0.000	......	0.000	0.000	......	0.000	0.000
7 800	......	......	7 700	......	......	7 800	......	....	12 300	......	......
29 600	0.184	0.000	29 200	0.188	0.000	29 500	0.181	0.000	30 900	......	......
7 800	0.047	0.000	7 700	0.044	0.000	7 800	0.044	0.000	12 300	......	......
29 600	0.191	0.000	29 200	0.181	0.000	29 500	0.188	0.000	30 900	0.133	0.000
7 800	0.044	0.000	7 700	0.044	0.000	7 800	0.044	0.000	12 300	0.050	0.000
29 600	0.197	0.000	29 200	0.185	0.000	29 500	0.175	0.000	30 900	0.133	0.000
7 800	0.044	0.000	7 700	0.044	0.000	7 800	0.044	0.000	12 300	0.050	0.000
29 600	0.191	0.000	29 200	0.181	0.000	29 500	0.188	0.000	30 900	0.133	0.000
7 800	0.044	0.000	7 700	0.044	0.000	7 800	0.038	0.000	12 300	0.050	0.000
29 600	0.191	0.000	29 200	0.181	0.000	29 500	0.188	0.000	30 900	0.133	0.000
43 600	0.297	0.006	43 100	0.282	0.006	43 500	0.288	0.000	34 000	0.144	0.000
......	......	......	44 600	0.300	0.009	......	......	......	37 000	0.162	0.003
46 800	0.335	0.016	46 200	0.313	0.013	46 600	0.313	0.000	40 100	0.181	0.006
48 300	0.360	0.028	47 700	......	0.016	48 200	0.319	0.000	43 200	0.206	0.011
49 900	peeling 0.385	0.041	49 300	......	0.019	49 800	0.331	0.003	44 800	0.219	0.016
51 400	0.422	0.247	50 800	0.357	0.022	51 300	0.350	0.006	46 300	......	0.019
......	......	......	52 300	0.369	0.025	52 900	0.363	0.009	47 800	0.238	0.028
......	......	......	53 900	peeling 0.394	0.038	54 400	0.288	0.013	49 400	0.256	0.031
......	......	......	55 400	0.419	0.056	56 000	0.400	0.022	52 500	peeling 0.288	0.050
......	......	......	57 000	0.457	0.088	57 500	peeling 0.419	0.034	55 600	0.344	0.094
......	......	......	......	......	......	59 100	0.588	0.200	......	......	....
Ultimate strength	88 900			90 800			90 800			96 900	
Order of testing...	1			3			2			4	

The stretch and set were measured in a length of 15 ft. for $B\,1$, $B\,2$, and $B\,3$, and 10 ft. for $B\,4$; they are expressed in decimals of an inch.

The pointer of the extensometer did not assume permanent zero upon the removal of the load until after a load of 475 000 lb. had been put on the 8-in. bars; the first readings, therefore, were discarded.

The first three readings for the 16-in. bar were discarded, for the same reason.

TABLE 56.—COMPARISON OF TENSILE TESTS ON SPECIMENS AND FULL-SIZED BARS.
EYE-BAR MATERIAL—NICKEL STEEL.

Cut from bar.	Dimension.	Load, in Pounds per Square Inch.		Elastic ratio. Per cent.	Percentage of Elongation in:				Reduction of area. Per cent.	Remarks.
		Yield point.	Ultimate strength.		8 in.	12 in.	10 ft.	20 ft.		
Specimen Tests.										
6 by 1 in..	1-in. round.	62 300	104 300	59.7	18.8	....	....	....	53.3	Original.
6 " 1 "..	1 " "	56 400	102 800	54.9	18.8	....	....	....	47.0	Annealed.
6 " 1 "..	1¼ by 1 in.	59 800	102 700	58.2	19.6	....	....	....	50.4	Original.
6 " 1 "..	1¼ " 1 "	54 000	99 000	54.5	19.2	....	....	....	48.3	Annealed.
8 " 2 "..	¾ " 2 "	58 100	98 600	58.9	21.8	....	....	....	50.9	Original. Bars *B*1 and *B*2. Yield point of No. 11 omitted.
8 " 2 "..	¾ " 2 "	52 800	95 700	55.2	20.7	....	....	....	47.8	Annealed. " " " "
8 " 2 "..	¾ " 2 "	55 200	99 500	55.5	19.8	....	....	....	47.6	Original. Bar *B*3.
8 " 2 "..	¾ " 2 "	51 900	94 200	55.0	21.4	....	....	....	49.8	Annealed. " "
16 " 2 "..	¾ " 2 "	59 100	103 600	57.0	20.8	....	....	....	47.2	Original.
16 " 2 "..	¾ " 2 "	55 600	103 200	53.9	19.4	....	....	....	44.3	Annealed.
Full-Sized Tests.										
Mark.										
B 1.........	6 by 1 in.	58 700	99 000	59.3	....	24.2	12.5	11.0	40.6	
B 2.........	6 " 1 "	55 400	101 500	54.6	....	20.3	11.3	9.5	37.8	
B 3.........	6 " 1 "	55 400	103 200	53.7	....	21.2	10.7	9.6	40.4	
B 4.........	6 " 1 "	56 100	100 300	55.9	....	25.8	10.6	9.5	46.1	
B 1.........	8 " 2 "	48 300	88 900	54.3	....	27.2	12.0	{ 19 ft. 10.5	{ 37.0	15 per cent. crystalline.
B 2.........	8 " 2 "	52 300	90 800	57.6	....	23.8	12.0	11.3	31.4	
B 3.........	8 " 2 "	54 400	90 800	59.9	....	26.8	14.2	12.5	41.8	
B 4.........	16 " 2 "	49 400	96 900	51.0	....	25.4	14.5	{ 14 ft. 13.6	{ 22.3	Fine crystalline.

TABLE 57.—TENSILE TESTS ON A. A. S. M. SPECIMENS. EYE-BAR 16 BY 2-IN.

6 and 8-in. bars—Heat No. 17 749;

Cut from:	Original.		Fractured section, in inches.	Load, in Pounds per Square Inch.		Elastic ratio. Per cent.
	Section, in inches.	Area, in square inches.		Yield point.	Ultimate strength.	
6 by 1-in.	Diameter = 0.959	0.722	Diameter = 0.655	62 300	104 300	59.7
"	Diameter = 0.962	0.727	Diameter = 0.700	56 400	102 800	54.9
"	1.255 by 0.985	1.236	0.900 by 0.665	....	102 400	....
"	1.250 " 0.995	1.244	0.930 " 0.695	....	98 600	....
"	1.265 " 0.985	1.246	0.925 " 0.675	62 600	101 300	61.8
"	1.255 " 0 995	1.249	0.930 " 0.680	55 200	98 700	55.9
"	1.250 " 0.985	1.231	0.905 " 0.685	56 900	104 500	54.5
"	1.245 " 0.990	1.233	0.925 " 0.700	52 700	99 800	52.8
8 by 2-in.	0.760 " 1.985	1.509	0.505 " 1.465	*61 600	98 700	62.4
"	0.780 " 1.990	1.552	0.515 " 1.490	51 500	94 500	54.5
"	0.755 " 1.980	1.495	0.475 " 1.480	54 700	93 800	58.3
"	0.755 " 1.975	1.491	0.510 " 1.480	51 000	92 600	55.1
"	0.750 " 1.990	1.493	0.500 " 1.470	*60 900	99 900	61.0
"	0.765 " 1.985	1.518	0.530 " 1.510	54 000	96 800	55.8
"	0.755 " 1.985	1.499	0.515 " 1.495	58 700	101 800	57.7
"	0.755 " 1.990	1.502	0.555 " 1.550	54 600	98 900	55 2
"	0.740 " 1.985	1.469	0.520 " 1.535	59 900	103 500	57.9
"	0.760 " 1.985	1.509	0.510 " 1.510	53 000	96 100	55.2
"	0.765 " 1.980	1.515	0.525 " 1.460	50 400	95 400	52.8
"	0.755 " 1.980	1.495	0.500 " 1 465	50 700	92 300	54.9
16 by 2-in.	0.760 " 2.030	1.543	0.525 " 1.570	59 600	104 000	57.3
"	0.760 " 2.025	1.539	0.550 " 1.575	55 900	103 700	53.9
"	0.755 " 2.025	1.529	0.515 " 1.550	58 500	103 100	56.7
"	0.760 " 2.025	1.539	0.540 " 1.550	55 200	102 700	53.8

Pieces cut out from 6 by 1-in. Bar, *B*1, were turned to 1 in. in diameter and used for
" " " " 6 by 1-in. Bars *B*2, 3 and 4, and piece *TSLEB*32, were tested on
" " " " 8 by 2-in. and 16 by 2-in. bars, except *TSLEB*32, were tested on

* Autographic curves are abnormal at yield point.

† Broke 1⅛ in. from end gauge mark.

MATERIAL—NICKEL STEEL—CUT FROM 6 BY 1-IN., 8 BY 2-IN., AND EYE-BARS.

16-in. bars—Heat No. 17 673.

Percentage of Elongation in:				Reduction of area.	Character of fracture.	Speed of Breaking.		Location of specimen.	Remarks.
2 in.	4 in.	6 in.	8 in.			At yield point.	At break.		
37.0	26.5	21.7	18.8	53.3	S ½ Cup.	Very slow.	1 in. in 2 min.	All	Original.
35.0	25.5	21.0	18.8	47.0	S ⅓ Cup.	" "	"	half	Annealed.
40.5	26.5	21.3	19.0	51.6	S Cup.	1 in. in 10 min.	1 in. in 1 min.	way	Original.
37.5	25.5	21.2	18.5	48.1	S ¾ Cup.	"	"	between	Annealed.
41.0	28.3	23.0	20.6	49.9	S Ang.	"	"	edge	Original.
38.5	27.0	22.3	19.5	49.4	S Cup.	"	"	and	Annealed.
40.5	26.5	21.7	19.3	49.6	S Ang.	"	"	middle	Original.
40.0	28.3	22.7	19.5	47.4	S ¾ Cup.	"	"	of bar.	Annealed.
40.0	30.0	23.3	20.3	51.0	S Ang.	1 in. in 20 min.	1 in. in 2 min.	Middle.	Original.
40.0	28.5	23.3	†20.0	50.6	S Cup.	"	"	"	Annealed.
35.0	31.0	25.7	22.4	53.0	S Irreg.	"	"	Edge.	Original.
45.0	31.5	25.3	21.8	49.4	S Ang.	"	"	"	Annealed.
44.0	31.2	25.7	22.4	50.8	S Ang.	"	"	Middle.	Original.
42.0	29.0	23.3	20.3	47.3	S Ang.	"	"	"	Annealed.
42.0	30.5	25.0	22.0	48.6	S Cup.	"	"	Edge.	Original.
37.0	28.5	23.7	20.8	43.7	S Cup.	"	"	"	Annealed.
39.0	26.0	20.0	†17.5	45.7	S Ang.	"	"	Middle.	Original.
40.0	29.5	24.0	20.8	49.0	S Ang.	"	"	"	Annealed.
45.5	32.0	25.8	22.0	49.5	S Cup.	1 in. in 10 min.	1 in. in 1 min.	Edge.	Original.
45.0	31.0	25.3	22.0	51.0	S Ang.	1 in. in 20 min.	1 in. in 2 min.	"	Annealed.
42.0	29.5	24.3	20.5	46.6	S Cup.	"	"	Middle.	Original.
34.0	24.5	22.0	19.3	43.0	S Cup.	"	"	"	Annealed.
42.0	29.5	24.0	21.0	47.8	S Ang.	"	"	Edge.	Original.
37.0	27.5	22.3	19.5	45.6	S Ang.	"	"	"	Annealed.

Coefficient of Elasticity determination.
150 000-lb. Riehle machine at Riehle Bros. Co.
200 000-lb. Olsen machine at Drexel Institute.

TABLE 58.—BENDING TESTS ON PLAIN SPECIMENS CUT FROM 6 BY 1-IN., 8 BY 2-IN., AND 16 BY 2-IN. EYE-BARS.
EYE-BAR MATERIAL—NICKEL STEEL.
Tests Made on Hydraulic Bending Machine, Pencoyd, Pa.

Cut from eye-bar:	Heat treatment.	Angle of bend. in degrees.	2 × (Radius of Bend): Inches.	2 × (Radius of Bend): "t"	Opening at 1 in. from inside edge, in inches.	Remarks.
6 by 1-in.	Original.	191	15/18	0.9	1 3/16	No cracks.
"	Annealed.	190	15/16	0.9	1 3/16	Skin broken.
"	Original.	192	15/16	0.9	1 3/16	Crack 1 in. long, another ¾ in. long.
"	Annealed.	191	1	1.0	1 1/4	Skin broken.
"	Original.	191	1 1/16	1.1	1 3/8	Three small cracks.
"	Annealed.	188	13/16	0.8	1	No cracks.
"	Original.	194	1 1/4	1.3	1 1/2	One small crack.
"	Annealed.	190	15/16	0.9	1 3/16	No cracks.
8 by 2-in.	Original.	175	2	1.0	2	"
"	Annealed.	174	2 1/8	1.1	2 1/4	"
"	Original.	173	2	1.0	2 1/8	"
"	Annealed.	171	1 3/4	0.9	1 7/8	"
"	Original.	175	2 1/4	1.1	2 1/4	"
"	Annealed.	173	1 5/8	0.8	1 11/16	One slight crack in corner only.
16 by 2-in.	Original.	173	2 5/8	1.3	2 1/2	No cracks.
"	Annealed.	177	2 1/2	1.3	2 3/8	"

"t"=thickness of material.
6 and 8-in. bars were rolled from Heat No. 17 749; 16-in. bars from Heat No. 17 673.

APPENDIX B. (PART I.)

TABLE 59.—CHEMICAL ANALYSES AND SPECIMEN TEST RESULTS OF MATERIAL FROM WHICH FULL-SIZED TESTS MET SPECIFICATIONS. *B*-1 200—BLACKWELL'S ISLAND BRIDGE—NICKEL-STEEL EYE-BARS.

Heat No.	Chemical Analysis.					Unannealed.				Annealed.			
	C.	P.	S.	Mn.	Ni.	Elastic limit.	Ultimate strength.	Elonga-tion.	Reduction.	Elastic limit.	Ultimate strength.	Elonga-tion.	Reduction.
15 119	0.36	0.010	0.027	0.75	3.36	56 980	100 500	18.7	35.9	50 450	94 080	27.0	50.5
15 121	0.32	0.014	0.029	0.65	3.36	55 740	95 260	21.2	37.1	50 880	87 740	26.2	50.9
15 121	0.32	0.014	0.029	0.65	3.36	55 740	95 260	21.2	37.1	50 880	87 740	26.2	50.9
15 121	0.32	0.014	0.029	0.65	3.36	55 740	95 260	21.2	37.1	50 880	87 740	26.2	50.9
16 122	0.37	0.016	0.031	0.70	3.34	57 500	99 350	19.5	30.8	55 100	97 340	19.7	54.4
16 122	0.37	0.016	0.031	0.70	3.34	57 500	99 350	19.5	30.8	55 100	97 340	19.7	54.4
16 068	0.32	0.015	0.030	0.65	3.22	57 230	96 510	20.0	34.5	51 840	87 470	24.2	44.3
1 088	0.38	0.012	0.029	0.60	3.36	55 200	100 400	20.0	29.2	53 360	90 830	21.2	48.5
16 245	0.37	0.010	0.022	0.74	3.36	60 650	106 900	17.5	25.6	54 020	97 560	20.0	41.0
16 230	0.40	0.013	0.026	0.67	3.34	57 890	104 000	16.2	25.8	53 720	91 520	25.0	47.6
16 257	0.38	0.012	0.033	0.72	3.26	59 300	104 500	16.2	27.4	53 160	97 290	23.7	45.3
16 313	0.43	0.014	0.033	0.75	3.28	56 180	119 900	15.0	28.6	51 260	105 200	21.0	43.9
2 139	0.38	0.010	0.028	0.76	3.28	58 550	102 200	20.0	32.0	54 080	96 200	18.2	46.1
1 105	0.36	0.010	0.030	0.73	3.30	58 940	105 500	19.2	28.3	51 860	86 400	25.0	44.9
16 225	0.41	0.010	0.034	0.70	3.36	60 280	110 100	16.2	34.0	54 620	101 900	20.5	41.6
1 114	0.41	0.012	0.037	0.62	3.52	59 270	104 400	14.0	19.0	52 400	97 400	21.2	29.7
16 227	0.34	0.016	0.032	0.70	3.28	55 380	104 100	20.0	34.6	55 640	91 060	21.2	29.0
16 227	0.34	0.016	0.032	0·70	3.28	55 380	104 100	20.0	34.6	55 640	91 060	21.2	29.0
16 222	0.45	0.013	0.025	0.70	3.26	64 780	113 500	18.1	33.5	50 840	101 200	21.25	46.4
16 255	0.41	0.012	0.026	0.80	3.30	62 780	115 400	16.2	31.8	52 120	104 200	20.0	34.2
16 308	0.34	0.010	0.030	0.66	3.28	62 940	107 200	18.7	25.4	54 220	93 360	22.5	48.6
16 248	0.41	0.012	0.030	0.80	3.28	66 360	116 000	15.0	28.8	60 160	104 800	22.5	42.8
16 260	0.40	0.010	0.030	0.78	3.28	55 240	114 000	16.0	28.9	53 780	103 600	18.5	37.2
16 316	0.38	0.010	0.028	0.70	3.40	55 610	95 140	20.0	38.7	52 560	97 940	22.5	44.1
10 120	0.41	0.010	0.035	0.70	3.40	64 420	119 800	16.2	23.7	55 020	103 200	21.2	42.2
2 161	0.38	0.010	0.035	0.74	3.26	57 560	106 200	22.5	45.4	50 220	100 900	21.25	33.6
1 115	0.43	0.016	0.025	0.75	3.32	63 960	110 100	16.0	28.9	54 250	104 200	18.70	38.2
16 290	0.39	0.010	0.035	0.75	3.36	61 260	108 300	17.2	30.8	54 010	95 540	23.7	51.3
16 272	0.40	0.012	0.025	0.79	3.25	66 760	111 600	14.0	20.1	55 860	102 600	20.0	40.2
16 263	0.36	0.012	0.028	0.74	3.46	65 250	107 050	17.5	27.4	48 040	94 070	23.7	45.0
16 281	0.44	0.010	0.026	0.72	3.28	63 200	116 500	15.0	20.8	55 600	105 800	22·5	32.2
16 280	0.42	0.011	0.039	0.69	3.26	67 640	117 000	13.7	21.5	51 460	101 700	17.5	31.5
16 292	0.43	0.010	0.035	0.80	3.36	66 080	112 100	14.5	25.3	55 140	98 320	21.2	42.5
16 239	0.37	0.010	0.030	0.65	3.46	60 400	101 600	18.7	33.3	53 950	92 160	21.2	38.7
16 239	0.37	0.010	0.030	0.65	3.46	60 400	101 600	18.7	33.3	53 950	92 160	21.2	38.7
16 303	0.44	0.010	0.025	0.80	3.76	60 620	113 400	17.2	26.4	55 850	101 500	21.0	43.9
16 238	0.37	0.012	0.033	0.85	3.30	65 430	110 200	18.7	32.1	50 440	96 640	15.0	22.3
10 096	0.40	0.010	0.028	0.62	3.36	55 760	94 000	20.0	34.4	49 670	88 380	21.2	40.1
16 231	0.39	0.012	0.031	0.80	3.30	63 900	110 100	16.0	28.9	55 400	104 300	20.0	30.0
2 145	0.36	0.015	0.030	0.75	3.28	56 140	105 400	17.0	35.2	55 360	97 700	24.2	50.5
16 226	0.37	0.012	0.030	0.78	3.26	57 840	109 100	18.7	32.4	55 630	96 740	23.7	47.2
16 285	0.45	0.011	0.030	0.78	3.26	67 120	112 400	15.0	27.4	52 080	103 400	20.0	37.4
16 236	0.36	0.012	0.035	0.65	3·28	61 860	104 000	16.2	25.0	52 200	91 720	21.2	45.2
16 322	0.42	0.010	0.028	0.77	3.30	66 080	116 200	16.7	31.0	58 060	102 100	20.0	30.9
16 223	0.45	0.011	0.030	0.75	3.28	68 300	118 300	15.0	24.2	54 250	110 800	21.0	38.8
16 224	0.38	0.011	0.033	0.72	3.36	63 440	105 400	15.0	17.3	52 130	95 620	18.25	20.3
10 115	0.40	0.012	0.032	0.87	3.26	64 480	105 200	14.5	18.8	55 980	105 900	20.0	35.3
1 110	0.38	0.010	0.034	0.72	3.46	60 000	102 200	18.7	28.2	51 540	94 800	21.2	41.0
1 123	0.43	0.916	0.030	0.72	3.32	62 260	110 300	17.2	30.9	62 140	102 100	21.2	41.0
16 258	0.37	0.010	0.026	0.73	3.28	57 420	104 000	18.7	32.9	53 000	94 200	23.7	40.6
10 111	0.40	0.010	0.030	0.64	3.34	62 360	110 200	17.06	32.2	56 050	98 100	21.2	37.6
2 165	0.37	0.013	0.037	0.80	3.28	62 600	113 200	17.5	15.9	51 600	99 480	20.0	46.9
10 116	0.37	0.014	0.029	0.70	3.36	61 570	100 300	20.0	31.8	54 270	84 100	25.0	49.1
16 311	0.37	0.010	0.033	0.75	3.36	62 450	105 800	18.7	34.4	54 880	97 620	21.2	45.5
16 256	0.40	0.012	0.022	0.73	3.28	67 700	112 200	14.5	24.1	53 250	101 600	22.5	47.3
16 851	0.38	0.010	0.028	0.64	3.48	56 230	100 500	20.0	32.7	53 980	95 400	26.2	53.0
16 861	0.39	0.010	0.030	0.69	3.34	55 900	103 800	18.7	30.7	52 900	94 440	24.7	48.5
14 566	0.41	0.011	0.031	0.65	3.60	60 900	105 900	19.7	38.3	56 560	97 020	23.5	40.1

TABLE 59.—(*Continued.*)

Heat No.	Chemical Analysis.					Unannealed.				Annealed.			
	C.	P.	S.	Mn.	Ni.	Elastic limit.	Ultimate strength.	Elonga-tion.	Reduction.	Elastic limit.	Ultimate strength.	Elonga-tion.	Reduction.
13 571	0.44	0.012	0.029	0.78	3.30	65 950	113 100	16.2	30.8	61 680	105 200	21.2	42.4
14 543	0.37	0.013	0.025	0.80	3.30	64 280	107 500	18.7	36.1	52 320	102 900	20.0	33.5
16 859	0.38	0.010	0.035	0.66	3.34	55 500	99 180	20.0	35.0	58 340	90 220	24.7	43.5
14 559	0.47	0.013	0.023	0.72	3.40	59 350	115 700	13.2	28.1	48 220	108 000	16.2	28.9
16 268	0.42	0.010	0.022	0.78	3.36	55 620	117 000	14.5	13.8	56 200	105 400	18.7	39.1
16 270	0.42	0.010	0.036	0.75	3.36	62 400	115 200	16.2	30.0	51 000	101 000	21.5	45.6
14 545	0.40	0.010	0.036	0.66	3.26	55 680	104 100	21.2	36.9	50 800	95 060	22.5	45.1
16 305	0.44	0.010	0.025	0.73	3.38	60 740	116 700	17.5	29.5	52 040	103 300	22.0	41.3
14 555	0.46	0.010	0.036	0.65	3.36	57 300	116 400	15.0	22.4	62 620	108 200	20.5	38.2
14 530	0.40	0.010	0.029	0.63	3.30	55 220	101 400	23.2	43.7	50 100	91 560	23.2	54.5
14 558	0.39	0.022	0.031	0.85	3.40	65 750	110 400	17.0	31.1	58 820	101 400	22.2	50.3
12 738	0.41	0.010	0.034	0.72	3.54	62 020	102 900	18.7	32.8	52 960	95 300	22.5	50.5
14 552	0.44	0.016	0.031	0.63	3.42	63 120	113 500	12.5	18.6	56 720	90 500	23.7	48.7
1 116	0.45	0.015	0.028	0.80	3.40	66 320	117 500	13.7	18.8	59 900	109 700	21.2	34.2
16 279	0.44	0.010	0.031	0.82	3.26	55 440	112 500	16.2	29.6	51 840	102 700	20.5	44.9
12 041	0.39	0.012	0.030	0.75	3.62	61 160	107 050	18.2	29.5	56 400	94 900	25.0	44.5
12 059	0.43	0.013	0.028	0.70	3.64	59 940	112 500	12.5	23.0	55 380	101 400	18.7	25.6
12 054	0.38	0.017	0.029	0.63	3.50	65 630	101 600	16.2	23.1	50 140	86 700	22.0	39.8
12 081	0.40	0.012	0.029	0.69	3.40	60 800	107 400	18.7	26.7	56 730	96 320	23.7	41.6
12 064	0.43	0.016	0.032	0.78	3.58	61 700	102 300	19.5	24.1	58 060	96 800	21.2	43.7
16 795	0.40	0.010	0.030	0.65	3.58	62 220	103 000	18.7	26.3	56 260	95 200	23.7	46.7
12 055	0.38	0.010	0.037	0.69	3.56	62 320	102 800	18.5	28.5	56 360	88 140	25.0	49.5
12 049	0.38	0.010	0.030	0.78	3.59	64 170	103 800	17.0	27.7	57 160	89 950	25.0	52.7
16 306	0.42	0.010	0.030	0.78	3.32	60 480	109 800	17.5	33.6	50 100	98 960	21.2	44.3
16 320	0.42	0.010	0.027	0.75	3.32	62 600	110 700	16.2	32.0	53 720	95 740	21.2	42.8
12 037	0.39	0.012	0.030	0.69	3.46	61 200	105 000	19.7	28.5	61 510	93 010	28.0	50.8
16 262	0.42	0.010	0.022	0.73	3.39	61 300	116 500	15.0	31.3	54 450	104 200	21.2	40.4
16 328	0.36	0.010	0.031	0.76	3.32	64 490	108 850	17.0	19.4	58 220	95 460	21.7	38.9
16 330	0.38	0.014	0.038	0.70	3.32	60 950	106 900	18.7	31.4	57 200	96 190	21.2	44.1
14 548	0.41	0.010	0.036	0.73	3.40	61 800	108 650	20.0	38.7	59 180	102 350	22.5	46.2
16 791	0.39	0.010	0.030	0.69	3.52	62 800	104 600	20.0	32.2	55 400	93 300	25.0	48.7
12 014	0.39	0.012	0.032	0.66	3.42	55 760	102 900	20.0	28.5	53 200	92 500	21.2	46.3
12 080	0.40	0.010	0.025	0.75	3.50	59 600	107 400	20.0	22.5	55 190	96 970	24.5	40.9
14 743	0.42	0.010	0.035	0.71	3.54	59 930	111 800	16.2	27.4	56 380	100 600	23.2	47.1
14 742	0.42	0.013	0.032	0.65	3.46	56 500	104 900	18.7	29.3	54 080	96 240	25.7	45.1
16 309	0.37	0.010	0.032	0.79	3.26	57 160	107 700	20.0	38.5	51 260	97 820	26.2	49.8
14 747	0.38	0.012	0.030	0.69	3.56	60 140	100 400	20.7	37.1	57 060	92 560	27.5	49.5
12 737	0.41	0.012	0.034	0.65	3.26	57 600	104 500	18.7	31.4	56 940	93 480	22.2	47.0
12 776	0.40	0.010	0.029	0.66	3.62	59 880	102 900	18.7	29.6	52 980	94 460	22.5	45.1
12 707	0.41	0.010	0.029	0.68	3.30	62 640	106 700	15.0	31.2	58 740	97 200	23.7	49.3
1 444	0.38	0.012	0.034	0.63	3.30	55 340	98 700	20.0	29.7	52 280	86 720	23.7	49.6
12 261	0.42	0.010	0.027	0.70	3.50	61 900	103 500	21.2	33.5	52 940	96 560	25.0	42.9
12 013	0.41	0.012	0.032	0.68	3.40	58 860	103 500	17.5	27.3	48 250	88 240	25.0	45.3
12 003	0.37	0.010	0.034	0.66	3.64	56 760	102 700	20.0	28.0	53 960	95 720	23.7	40.7
12 884	0.45	0.010	0.027	0.65	3.60	55 580	109 300	16.2	26.2	59 060	100 500	21.7	41.6
12 193	0.39	0.012	0.029	0.70	3.48	60 600	100 400	22.0	38.2	51 810	92 040	23.7	45.6
12 740	0.38	0.013	0.032	0.69	3.34	61 020	105 400	17.5	34.0	52 800	93 460	22.5	50.7
12 062	0.37	0.011	0.032	0.75	3.46	59 720	101 300	20.5	40.9	52 640	89 160	23.7	47.7
12 340	0.40	0.012	0.027	0.62	3.36	62 840	109 400	15.0	25.8	53 940	94 120	23.2	42.0
12 709	0.41	0.010	0.029	0.68	3.36	61 000	103 706	18.7	29.8	48 330	96 540	25.0	46.1
12 340	0.40	0.012	0.027	0.62	3.36	62 840	109 400	15.0	25.8	53 940	94 120	23.2	42.0
11 434	0.40	0.012	0.037	0.65	3.64	58 340	100 300	19.5	31.3	53 700	89 920	24.2	41.9
12 359	0.39	0.010	0.032	0.74	3.50	64 920	104 700	20.5	33.7	55 600	99 500	23.7	39.7
12 358	0.36	0.010	0.030	0.60	3.50	59 420	97 460	21.2	34.1	57 400	91 600	22.0	38.1
12 346	0.40	0.010	0.027	0.80	3.62	63 860	104 500	18.0	34.0	59 170	94 060	27.0	49.1
12 343	0.37	0.010	0.027	0.69	3.64	64 420	108 800	17.5	34.3	54 060	95 100	23.2	42.8
12 452	0.38	0.010	0.028	0.76	3.44	62 950	106 700	18.7	25.1	57 800	97 680	23.2	45.4
12 727	0.37	0.010	0.032	0.58	3.44	57 320	95 340	20.0	35.5	48 030	88 820	23.7	48.3
12 725	0.39	0.010	0.025	0.68	3.61	60 140	102 000	17.5	37.5	57 500	96 100	23.7	42.8
12 773	0.40	0.010	0.030	0.60	3.34	58 700	102 600	18.7	33.0	55 420	90 700	26.0	50.0
12 728	0.36	0.011	0.027	0.60	3.36	59 860	103 300	21.2	35.4	57 480	93 320	25.0	49.1
12 539	0.39	0.011	0.031	0.77	3.40	59 940	100 800	20.0	25.7	51 500	91 960	25.5	44.1
12 450	0.36	0.010	0.030	0.54	3.36	59 940	99 450	16.2	30.1	54 410	91 800	23.7	43.7
11 437	0.37	0.010	0.029	0.59	3.53	59 600	101 900	18.2	26.4	59 950	95 920	21.7	34.8
13 429	0.36	0.010	0.033	0.63	3.50	61 040	99 360	18.7	30.4	55 370	93 080	24.5	45.9
14 553	0.43	0.018	0.032	0.73	3.34	63 740	112 800	14.7	26.8	57 720	103 800	18.7	28.9
12 345	0.43	0.012	0.033	0.69	3.50	61 340	108 400	16.7	30.1	56 106	98 760	22.0	39.2

APPENDIX B. (PART II.)

TABLE 60.—RESULT OF TESTS THAT MEET SPECIFICATIONS. BLACKWELL'S ISLAND BRIDGE—NICKEL-STEEL EYE-BARS.

Heat No.	Section.	Percentage of nickel.	Elastic limit, in pounds per square inch.	Ultimate tensile strength, in pounds per square inch.	Percentage of elongation in 18 ft.	Reduction of area. Percentage.	Character of fracture.
15 119	16 × 2	3.36	54 260	87 710	6.80	12.3	Crystalline. Broke in head.
15 121	16 × 1 7/8	3.36	52 460	86 270	10.00	36.1	Silky.
15 121	16 × 1 15/16	3.36	50 590	85 841	11.10	40.5	95% silky.
15 121	16 × 1 7/8	3.36	51 780	88 280	10.32	28.6	Cup. 65% silky.
16 122	16 × 1 7/8	3.34	51 320	86 360	6.55	8.7	Crystalline. Broke in head.
16 122	16 × 1 7/8	3.34	52 740	90 080	7.00	11.8	Crystalline. Broke in head.
16 068	16 × 1 7/8	3.22	51 150	89 840	10.60	10.3	60% crystalline and 40% silky. Broke in head.
1 088	16 × 1 7/8	3.36	49 850	82 540	14.10	33.8	Silky: irregular.
16 245	16 × 1 7/8	3.36	49 810	87 970	14.77	39.6	" Cup.
16 230	16 × 2	3.34	51 280	90 120	9.22	....	Fine granular. Broke in head.
16 257	16 × 2	3.26	50 660	89 470	13.21	44.5	Silky cup.
16 313	16 × 2 1/8	3.28	50 072	98 980	10.55	38.9	Irreg. silky. Trace crystalline.
2 139	16 × 2 1/16	3.28	52 990	88 310	18.44	38.8	Silky. 90% cup.
1 105	16 × 1 7/8	3.64	48 051	85 535	13.05	35.3	" angular.
16 225	16 × 2 1/16	3.36	48 102	88 137	13.50	21.4	Irreg. silky and fine granular.
1 114	16 × 2	3.52	48 296	85 446	16.61	26.9	Silky 90% cup.
16 227	16 × 2	3.29	51 314	86 357	14.88	4.3	" 70% "
16 227	16 × 2	3.28	49 340	88 623	11.55	38.9	" angular.
16 222	16 × 2	3.26	51 020	95 392	12.11	40.4	" "
16 255	16 × 1 3/4	3.30	48 170	98 840	10.00	33.9	Fine granular.
10 308	16 × 1 15/16	3.28	49 315	82 410	17.61	44.7	Silky ⅛ cup.
16 248	16 × 1 7/8	3.28	49 150	91 160	16.16	29.1	" angular.
16 260	16 × 1 7/8	3.28	49 210	95 380	12.27	37.2	" "
16 316	16 × 1 15/16	3.40	48 150	82 950	12.11	39.7	" irregular.
10 120	16 × 1 7/8	3.40	48 400	84 500	12.83	35.9	" "
2 161	16 × 1 3/4	3.26	51 240	83 400	10.83	48.8	" cup.
1 115	16 × 2 1/16	3.32	49 100	98 350	11.22	36.7	" square. Trace granular.
16 290	16 × 1 7/8	3.36	49 360	92 080	16.00	44.0	" cup.
16 272	16 × 2 1/8	3.28	51 070	98 000	18.38	8.7	Broke in head.
16 263	16 × 2	3.46	53 000	87 600	15.94	44.2	Silky ½ cup.
16 281	16 × 2	3.28	49 290	94 890	10.55	28.3	Irreg. 80% crystalline, bal. silky.
16 280	16 × 2	3.26	49 040	96 700	12.16	27.4	Square. Near silky.
16 292	16 × 2	3.36	52 360	95 880	13.90	38.4	Silky cup.
16 239	16 × 1 7/8	3.26	48 130	84 160	12.66	21.8	75% crystalline, bal. irreg. silky.
16 239	16 × 1 7/8	3.26	48 150	89 020	14.00	36.5	Silky irregular.
16 303	16 × 1 7/8	3.26	48 110	86 000	9.66	28.7	75% crystalline, bal. silky.
16 238	16 × 1 3/4	3.30	50 090	91 000	15.94	41.3	Silky cup.
10 096	16 × 2	3.36	49 220	90 020	11.77	34.2	" irregular.
16 231	16 × 2 1/8	3.30	48 230	95 600	10.83	40.5	" "
2 145	16 × 2 1/16	3.28	53 110	90 670	13.77	6.8	" 25% angular, bal. square.
16 226	16 × 2	3.26	48 920	93 320	11.11	....	Too long to pull to fracture.
16 285	16 × 2 1/16	3.26	39 960	90 480	13.00	44.5	Irreg. 60% silky, bal. crystalline.
16 236	16 × 1 7/8	3.28	49 330	85 250	14.80	44.5	Silky angular.
16 322	16 × 1 15/16	3.30	48 270	93 500	9.06	41.4	" irregular.
16 223	16 × 2 1/16	3.28	48 250	91 010	13.33	41.9	Irregular silky.
16 224	16 × 2	3.36	49 850	89 260	12.16	....	Too long to pull to fracture.
10 115	16 × 2	3.26	49 200	95 760	13.16	32.7	Irregular silky.
1 110	16 × 2	3.46	49 090	90 390	17.11	34.2	Irreg. 50% silky, bal. granular.
1 123	16 × 1 7/8	3.32	50 530	91 810	11.05	17.0	Square granular.
16 258	16 × 1 7/8	3.28	48 260	83 630	11.50	31.8	Irregular silky.
10 111	16 × 1 7/8	3.34	51 070	85 490	9.44	....	Granular. Broke in head.
2 165	16 × 1 7/8	3.28	50 140	83 300	14.27	43.0	Silky irregular.
10 116	16 × 1 7/8	3.36	48 120	80 480	10.94	26.6	" angular.
16 311	16 × 1 7/8	3.36	50 260	85 980	9.77	38.8	" "

TABLE 60.—(*Continued.*)

Heat No.	Section.	Percentage of nickel.	Elastic limit, in pounds per square inch.	Ultimate tensile strength, in pounds per square inch.	Percentage of elongation in 18 ft.	Reduction of area. Percentage.	Character of fracture.
16 256	16 × 2	3.28	54 040	98 060	11.94	31.9	Silky irregular.
16 851	16 × 1 7/8	3.48	48 290	83 520	12.16	34.2	" "
16 861	16 × 1 7/8	3.34	48 060	86 180	9.11	44.5	" "
14 566	16 × 2 1/16	3.60	57 050	87 770	8.16	....	Granular. Broke in head.
13 571	16 × 1 7/8	3.30	48 520	90 030	10.55	43.4	Irregular. 40% silky, bal. gran.
14 543	16 × 2	3.30	48 160	87 290	10.94	45.4	Irregular. 60% silky, bal. gran.
16 859	16 × 1 15/16	3.34	48 890	82 740	11.22	44.5	Silky angular.
14 559	16 × 2	3.40	48 120	90 780	9.77	37.2	20% silky, bal. crystalline.
16 268	16 × 2	3.36	52 970	87 520	6.05	....	50% silky, bal. crystalline. Broke in head.
16 270	16 × 1 15/16	3.36	48 050	90 920	10.33	44.4	Silky irregular.
14 545	16 × 1 13/16	3.26	48 110	84 970	9.16	32.2	Irregular. 30% silky, bal. crystalline.
16 305	16 × 2	3.38	53 200	88 030	6.00	....	Granular. Broke in head.
14 555	16 × 1 7/8	3.36	52 870	95 010	7.55	....	Granular. Broke in head.
14 530	16 × 1 15/16	3.30	51 340	86 050	7.33	....	30% silky, bal. crystalline. Broke in head.
14 558	16 × 1 7/8	3.40	48 200	91 920	13.66	38.8	Silky angular.
12 738	14 × 1 11/16	3.54	50 400	85 470	13.05	43.8	Silky ⅓ cup.
14 552	14 × 1 11/16	3.42	48 270	91 760	10.33	37.9	Silky irregular.
1 116	16 × 2 1/16	3.40	48 240	93 900	9.72	32.8	20% silky, bal. cryst.
16 279	16 × 2 1/16	3.36	56 380	94 430	12.05	33.5	Silky irregular.
12 041	16 × 1 13/16	3.62	58 000	89 860	14.27	32.7	" angular.
12 059	16 × 1 13/16	3.64	48 980	88 870	9.44	34.9	" irregular.
12 054	16 × 1 15/16	3.50	50 080	86 170	10.55	41.6	70% silky, bal. cryst.
12 081	16 × 1 13/16	3.40	51 320	87 980	7.22	....	30% " " " Broke in head.
12 064	16 × 1 13/16	3.58	52 160	92 150	10.00	33.3	Silky irregular.
16 795	16 × 2 1/8	3.58	48 290	86 130	10.94	41.9	" cup.
12 055	16 × 1 15/16	3.56	48 140	87 400	10.88	41.1	" irregular.
12 049	16 × 1 3/4	3.54	52 210	87 620	11.60	43.9	" ½ cup.
16 306	16 × 2 1/8	3.32	48 230	82 650	8.44	27.4	" square.
16 320	16 × 2 1/8	3.36	48 310	84 470	9.88	35.5	" irregular.
12 037	16 × 2 1/8	3.46	52 490	89 560	6.44	....	Granular. Broke in head.
16 262	16 × 2 1/16	3.34	48 240	87 680	13.88	43.5	80% silky, bal. cryst.
16 328	16 × 2 1/8	3.32	48 210	82 730	9.11	38.2	Silky irregular.
16 330	16 × 2 1/8	3.32	48 170	81 670	12.44	45.8	" ½ cup.
14 548	14 × 1 7/8	3.40	53 330	91 890	8.83	....	Crystalline. Broke in head.
16 791	16 × 2 1/8	3.52	49 230	82 250	9.16	46.3	Silky irregular.
12 014	16 × 2	3.42	48 040	85 580	10.27	38.9	" "
12 080	16 × 2	3.50	48 010	87 280	10.11	39.3	50% silky and bal. cryst.
14 743	16 × 1 7/8	3.54	55 900	98 000	10.77	34.2	10% " " "
14 742	16 × 2 1/16	3.46	51 350	90 180	9.38	....	Fine crystalline. Broke in head.
16 309	16 × 2 1/8	3.26	55 930	92 800	7.28	....	Crystalline. Broke in head.
14 747	16 × 1 3/4	3.56	54 112	87 810	11.05	42.0	Silky irregular.
12 737	16 × 2	3.46	55 690	86 320	12.33	33.9	" angular.
12 776	16 × 2	3.62	54 820	90 220	7.22	....	Granular. Broke in head.
12 707	16 × 2 1/16	3.30	56 270	89 130	6.50	....	Crystalline. " " "
1 444	16 × 2	3.30	52 860	83 020	11.27	28.2	Angular. ½ silky and bal. crystalline.
12 261	16 × 2 1/8	3.50	50 000	85 500	12.55	42.9	Silky irregular.
12 013	16 × 2 1/8	3.40	52 050	86 170	6.88	....	Granular. Broke in head.
12 003	16 × 2 1/8	3.64	54 390	88 970	10.38	40.2	Silky irregular.
12 884	16 × 2 1/8	3.44	49 160	80 360	10.00	37.2	" angular.
12 193	16 × 1 15/16	3.48	48 240	81 330	11.16	34.4	" irregular.
12 740	16 × 2	3.34	48 460	87 190	12.77	42.0	" angular.
12 062	16 × 1 15/16	3.54	60 570	87 790	11.00	....	Granular. Broke in head.
12 340	14 × 1 15/16	3.62	48 260	87 560	12.05	45.3	Silky angular.
12 709	16 × 2	3.36	51 130	87 670	8.50	....	80% crystalline. Broke in head.
12 340	16 × 2 1/8	3.36	52 080	89 280	7.66	....	Granular. Broke in head.
11 434	16 × 1 15/16	3.64	51 170	87 420	6.83	....	" " " "

TABLE 60.—(*Continued.*)

Heat No.	Section.	Percentage of nickel.	Elastic limit, in pounds per square inch.	Ultimate tensile strength, in pounds per square inch.	Percentage of elongation in 18 ft.	Reduction of area. Percentage.	Character of fracture.
12 359	16 × 1 15/16	3.50	57 260	96 930	6.00	....	Granular. Broke in head.
12 358	14 × 1 15/16	3.50	48 140	83 550	11.66	35.5	Irregular. 60% cryst. and bal. silky.
12 346	14 × 1 7/8	3.62	58 210	97 920	12.16	42.7	Silky ½ cup.
12 343	16 × 1 3/4	3.64	57 500	94 260	6.00	....	Granular. Broke in head.
12 452	14 × 1 3/4	3.44	56 000	95 090	9.06	32.0	Silky irregular.
12 727	16 × 2 1/16	3.44	52 440	95 420	6.55	....	75% silky, bal. gran. Broke in head.
12 725	16 × 2 1/16	3.64	50 320	85 910	9.78	44.7	Silky irregular.
12 773	16 × 2 1/16	3.34	49 030	83 840	10.94	46.4	" cup.
12 728	16 × 2 1/16	3.36	56 680	85 800	14.28	....	" irregular. Broke in head.
12 539	14 × 1 15/16	3.40	52 270	89 620	9.00	37.8	" cup.
12 450	14 × 1 15/16	3.36	52 920	86 680	9.11	4.1	" angular.
11 437	16 × 2 1/8	3.53	58 030	89 070	6.05	....	Granular. Broke in head.
12 429	16 × 2	3.50	50 060	89 740	12.83	33.4	Silky square.
14 553	14 × 1 7/8	3.34	53 090	87 560	6.55	....	Crystalline. Broke in head.
12 345	16 × 2	3.50	51,020	86 420	14.77	41.6	50% silky and 50% fine crystalline.

DISCUSSION.

Mr. Fowler.

CHARLES EVAN FOWLER, M. AM. SOC. C. E. (by letter).—The engineering profession and the metallurgists engaged in the manufacture of structural steel are certainly to be congratulated upon the extensive and careful investigations made by Mr. Waddell into the use of nickel steel for structural purposes, and also upon the work that has been done by F. C. Osborn, M. Am. Soc. C. E., in assisting in this valuable work.

The writer was one of the first to adopt the use of soft medium steel for structural purposes, as covered by "General Specifications for Steel Roofs and Buildings," and is still of the belief that, for a great many years, nothing better will be found for short-span bridges.

The small saving in cost by using either medium steel or nickel steel, would hardly be a valid reason for making a change, and, even while it might be considered advisable by some engineers to use nickel steel for medium-length spans, it will undoubtedly be many years before soft medium and medium steel are entirely displaced in the building of short and medium-length spans.

The great ease with which members can be fabricated from soft steel, and the reliability of the structures with only the usual amount of fair reaming in the shop, are almost unanswerable arguments in favor of continuing the use of this metal for short spans.

It is to be presumed from the data in the paper that the difference in cost of fabrication, as between medium steel fully reamed, and nickel steel, is very small; so that, for spans of considerable length, and for very long spans, there can be little question that if nickel steel proves to be as reliable as the tests stated in the paper would indicate, it will come into extensive use in the near future, more especially as this would enable the engineer to use spans of several hundred feet greater length than is possible at present. The writer, however, has not checked Mr. Waddell's figures, on which the cost of long spans was compared, but it would seem doubtful if it would be possible to make an increase to the extent of 500 ft., mainly on account of so many other factors than the mere cost of the metal entering into the cost of such structures.

The data available for reference by the writer, in addition to this paper, would seem to indicate that the machining of nickel steel would be much more difficult than that of ordinary carbon steel, although, with the small percentages discussed in the paper, for actual use this would not seem to be a very serious matter, and could only be determined definitely by actual experience in the shop.

The tests also seem to indicate that the opinions held in the past as to nickel steel are in the main correct, and that the effect of the nickel is quite uniform. Would it not be well, however, to make a

careful investigation of acid nickel steel, as there is no question that for a high grade of steel for long spans, the acid process is somewhat better than the basic. Mr. Fowler.

With reference to the future cost of nickel, it would hardly seem likely that even with the additional deposits which have been discovered, the cost would be very greatly reduced, and it would be well to know whether or not the author has investigated the use of ferruginous nickel for making the alloy, which would be just as effective, and of very much less cost.

It will undoubtedly be necessary to make a great many additional tests before nickel steel can be put into general use, and it is to be hoped that some of the large steel manufacturers will undertake this work, which, to the necessary extent, can hardly be carried on at private expense.

M. F. Brown, M. Am. Soc. C. E. (by letter).—One of the most important facts to be determined in connection with the use of nickel steel for bridges is the proper proportion of nickel and other elements to make the resultant alloy the most adaptable for the purpose intended. Dr. Waddell is evidently of the same opinion, but it seems to the writer that his paper does not give enough prominence to this fact. The difficulties in the way of a private individual who may undertake to determine these proportions are apparent; and certainly few, if able, would be willing to devote the time and money necessary to such a solution, as this will hardly be found except after the lapse of years. It is probable that nickel steel, as a material for bridge spans of ordinary length, if it is ever commonly used for such purposes, will have to pass through some such period of development as has structural steel during the last fifteen or twenty years, and be specified with proportions giving elastic limits ranging from those but slightly greater than carbon steel up to that as high as possible, consistent with fabrication under refined shop methods. From these extremes will probably issue a generally accepted material adaptable to ordinary shop methods of manufacture. Whether or not this material will be similar to that proposed by the author is impossible to determine; but it seems to the writer that just because two melts of steel having practically identical proportions give a product of good quality which can be readily fabricated into structural members, it is a little hasty to conclude that "it is not likely that any great improvement in the characteristics of the future plate-and-shape steel, as compared with those of these melts, will be affected." Mr. Brown.

The author's tests are well chosen to show the suitability of the material for the purpose intended, and certainly indicate the fitness of nickel steel as a material for bridge construction in spans of ordinary length, these being of the greatest interest to most engineers. Long spans, while of great interest in themselves, do not come within the

Mr. Brown. range of experience of the majority of engineers, and can be treated in a measure apart, being of such bulk that special material more particularly adaptable to them may be easily obtained if desired.

The tests for resilience are somewhat surprising, the general impression being that nickel steel has considerably more resilience than ordinary carbon steel; but the difference is not enough to prejudice its use. The corrosion tests are interesting as showing some comparative measure, with steel, of the action of gases, etc. It would have been interesting if these tests could also have been compared on an iron basis.

The writer is not disposed to agree with the author that the results of his investigation make it "practicable to write specifications for nickel-steel bridges which will possess the same strength, rigidity, and general excellence of design as the best carbon-steel bridges that are being built to-day." Anything like a general specification at the present time is attempting too much. The author's tests, valuable as they are in affording an indication of the use of the material, are too few to serve as a basis upon which to write a complete general specification. If, however, as seems probable, the author intends these specifications to serve as a guide only until further information is available, then they can serve a valuable purpose. As applied to the material as specified, the only criticism the writer offers concerns the compression formulas, which are based upon only six tests of full-sized members. This information appears to be too meager to develop a compression formula which will inspire confidence in the minds of conservative designers, and, until further tests are available, prudence would suggest reducing these units to an undoubtedly safe value.

An enormous amount of labor is indicated in the preparation of the diagrams giving weights of bridges and their comparative costs with all carbon steel and mixed nickel and carbon steel. It seems almost a pity that the author should have gone into this so deeply when it is considered that the specifications for the two materials can hardly be expected to maintain the relations existing when the diagrams were prepared. However, they will illustrate their purpose, and the profession is in a position to appreciate them.

These remarks are not intended in a spirit of criticism of this most valuable and timely paper, and the writer wishes to add his word of appreciation of the spirit which makes such a paper possible and available to other and less gifted engineers.

Mr. Bell. H. P. Bell, M. Am. Soc. C. E. (by letter).—Dr. Waddell's paper is likely to be of great service, not only to the profession, but also to the general public.

When it is considered that manufacturers can produce steel wire with an ultimate tensile strength of 100 tons per sq. in., and plate-

and-shape steel of about one-third the same tenacity, it is evident that the makers of the latter should be encouraged to improve its quality as much as possible. Mr. Bell.

It is a common expression that "it is the last straw that breaks the camel's back," and this aphorism comes forcibly home to the engineer who is designing a long-span bridge for modern double-track railway and other kinds of traffic combined. The use of nickel steel, as suggested by Dr. Waddell, may solve some problems not otherwise easy of solution.

As soon as engineers begin to manifest practically a desire to have a superior quality of nickel steel for bridge purposes, the makers themselves will take up the subject in earnest. The demand must come, before the quality of the supply will be as good as it can be made, and it probably will take some time to find out how to manufacture a superior kind of nickel steel for structural purposes. It is not to be supposed that an improvement once begun will be suddenly arrested. Upon the face of the facts already ascertained, there is something puzzling about the making of good nickel steel for bridge purposes—something that requires continued action among the makers—to throw light upon apparent inconsistencies, and to clear the way to the perfection that is likely to be attainable with due persistence.

Not many years ago metallurgists found great difficulty in the reduction of refractory ores, but, being persistent in their efforts, they finally succeeded to a very large extent, but this improvement may continue for a long time still.

"Resistance to cracking, a property to which the name of non-fissibility has been given, is shown more remarkably as the percentage of nickel increases. Bars of 27% nickel illustrate this property. A 1¼-in. square bar was nicked ¼ in. deep and bent double on itself without further fracture than the splintering off, as it were, of the nicked portion. Sudden failure or rupture of this steel would be impossible; it seems to possess the toughness of rawhide with the strength of steel. With this percentage of nickel the steel is practically non-corrodible and non-magnetic. The resistance to cracking shown by the lower percentages of nickel steel is best illustrated in the many trials of nickel-steel armor."*

In a table on the page from which the above quotation is taken, there is the record of two specimens of 3¼% nickel steel (bars forged down from a 6-in. ingot to ⅝ in. diameter, with conical heads for holding), the first with an ultimate tensile strength of 276 800 lb. and the second with 246 595 lb. per sq. in. On looking at the reduction of area and elongation, it is seen that neither rises above 6 per cent.

As these ingots were not rolled but forged down, it looks as if the amount of work put upon them, while increasing the tensile strength,

* See "The Mechanical Engineers' Pocket-Book," by William Kent, p. 408.

Mr. Bell. had impaired the reduction and elongation. It is in questions of this kind that manufacturers can be helpful to engineers, and it seems to the writer that engineers in specifying what they require, should not fall below the standards of this paper, but rather, if anything, exceed them, because the latter policy would put the manufacturers on their mettle. If it be assumed, in the first instance, that nothing better can be got than that which has been made already, progress and improvement will be slow. Suppose that a large bridge was to be tendered for and a notice such as the following was given:

Every contractor must furnish with his tender ——— bars of nickel steel of such and such size for testing purposes; and other things being equal, the tender possessing the strongest steel will have the best chance of securing the contract, which will not necessarily be let to the lowest bidder, but the contractor who is awarded the work shall bind himself to manufacture steel of at least equal quality with the samples tested.

If such a notice (or one more carefully drawn) were used, the probability is that a superior class of nickel steel could be had in the shortest time possible.

It was not to be expected that a preliminary inquiry, however exhaustive, into a subject of such magnitude and importance to the profession and the public, would cover all the ground necessary to be gone over. This is a matter of time and probably also of expenditure, large enough to awaken the sympathy, and engage the active assistance, of all those who may be commercially interested in the making of nickel steel for structural purposes.

Lately the writer became aware of the fact that a three-hinged metal arch of 1 800 ft. span could be built of carbon steel for a metal weight of 35 000 lb. per ft.; no unit stress on main members of more than 15 000 lb. per sq. in., either in tension or compression; and upon other parts none greater than 18 000 lb. per sq. in.; width of the floor, 80 ft.; live load per foot, for double-track railway and other kinds of traffic, 16 000 lb.

If this same structure were to be built of equal strength of nickel steel, it would weigh very approximately three-fourths of the above, or about 26 250 lb., instead of 35 000 lb. per ft.

It will be evident to those who study Dr. Waddell's paper, that engineers could now safely build upon his specifications, and realize an important economy; and that as the improvement in the manufacture of nickel steel continued, a class of material, better than the manufacturers would probably admit of as practicable at the present time, would soon be secured.

The writer believes that Dr. Waddell and his associates have done a most meritorious piece of work at an opportune time. It was needful that someone should take up the subject, and it is doubtful if it could have fallen into hands more capable, generous, and painstaking.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—Experiments on a small scale to determine the unit stress per square inch on the raw material of new metals as they come from the mills is certainly highly desirable information to be used subsequently in the manufacture of bridge members; but this, strictly speaking, is not the material with which the bridge engineer has to deal. He is practically compelled to take the finished members as they come to him from the shops ready for erection. This is what he uses, and, therefore, the tests of full-sized members just as they come from the shops give him practical results of the highest importance, the intrinsic value of which cannot be over-estimated. Past experiments generally show "crippling" apparently somewhat inside of the so-called elastic limit. This, it seems, is undoubtedly due to the imperfections of detailing in shopwork which cannot be entirely avoided in any case. Shopwork detailing 20 years ago was very different from that of the present day, and, moreover, even to-day the work turned out under ordinary conditions may be vastly different from that turned out under a rush order. In the latter case the inspectors are often compelled to slur over many things which they would not think of passing otherwise, and the result is inferior shopwork, which reduces not only the strength, but also the life, of the whole structure. This is particularly true of built-up members requiring a large amount of riveting. Mr. Le Conte.

This being the case, it is more than interesting to study the author's experiments on well-built, full-sized members of nickel steel. These columns he speaks of as properly designed and properly manufactured. The results reported are certainly highly satisfactory, and are to be commended in every respect. These experiments seem to show that the engineer is now called upon to make his unit stresses a function of the shopwork, instead of a function of the so-called elastic limit. In the West, particularly where the matter of railroad transportation of finished members is a very serious item of expense, it often happens that it is much cheaper to order the different parts of bridge members all carefully matched and punched at the eastern shops. Then, after their arrival at their destination, they are assembled and riveted up by the western shops. As a result, there is generally a mixture of good and poor work, and no formula could come anywhere near fitting the facts in such cases.

Inasmuch as these experiments seem to show conclusively that the ultimate stress which finished bridge members will stand, when tested to destruction, is largely, if not entirely, a direct function of the quality of shopwork, it would seem to be entirely reasonable and proper that every important member of a bridge (tension as well as compression—or in some members both) should be tested systematically with the full loads called for in the specifications before they leave the shops. This requirement, although severe, would certainly develop the weak points in shopwork if any existed. Again, on the arrival of the struc-

Mr. Le Conte. tural iron at its destination, all members should be examined carefully to see that they have not been injured in transit. And finally, during erection, care should be taken that the members are not injured in handling by the erecting parties. It is an old saying, and a good one, that eternal vigilance is the price of good work.

The more the writer thinks over this paper, the more he is impressed with the fact that it is highly inadvisable to lower the standard unit stresses called for in all the standard specifications of the present day, because they are based primarily on long, painstaking, and widespread experience extending over many years of gradual and progressive improvement in shopwork. Furthermore, any attempt at reducing these unit stresses would be practically equivalent to offering a premium on cheap draftsmen and poor shopwork, both of which no engineer of experience in such matters would countenance.

Whenever the quality of shopwork for any reason begins to deteriorate, it is often extremely difficult to find out who is responsible for the unhappy results. Of course, true economy is always in order, and is certainly entitled to the highest respect, but true economy is also about the rarest article to find on the face of the earth. This arises from the fact that those in supreme charge rarely have any conception of the value of minute technical knowledge and experience in detailing shopwork, and very often discharge a capable and trustworthy man in order to make room for a cheaper substitute. This is too often the case, and, as a result, they never seem to wake up or to realize fully the facts of the situation until some heavy bridge member, being handled by a yard derrick, buckles up right before their eyes. In shopwork, especially, the honest laborer, the world over, is always worthy of his hire.

This paper is extremely instructive, and shows that the author is a master in the details of his line of business. It is written in a most fascinating way, and the happy results reported are so alluring that one is naturally and unavoidably drawn toward his side of the case. He shows conclusively the great economy in the use of nickel steel for bridge work in general.

The enormous increase in the available length of maximum span—some 500 to 600 ft.—speaks volumes for all future designs in nickel-steel bridgework and structural materials of all kinds. Moreover, nearly 20% reduction in the cost of long spans will certainly put new life into the whole business, and will produce a great and lasting public benefit, the full measure of which cannot be comprehended by the average man.

Mr. Hatt. W. K. Hatt, Assoc. M. Am. Soc. C. E. (by letter).—It may seem anomalous that the nickel steel tested by the writer for Mr. Waddell should show a greater rupture-work (resilience) than the carbon steel,

which is more ductile. The values quoted by Mr. Waddell are as given in Table 61:* Mr. Hatt.

TABLE 61.—TENSILE IMPACT TESTS.

	Number tested.	Carbon.			Number tested.	Nickel (3.5%).		
		Maximum.	Minimum.	Average.		Maximum.	Minimum.	Average.
Elongation.........	5	32.0	31.0	31.5	4	19.00	13.00	16.5
Rupture-work......	5	1 910	1 540	1 736	3	2 300	1 960	2 198

The writer has consulted a paper of his, entitled "Tensile Impact Tests of Metals,"† and finds other comparative tensile impact tests of nickel and common soft machine steel, as shown in Table 62:

TABLE 62.

	Number tested.	Machine Steel.			Number tested.	3.15% Nickel Steel.		
		Maximum.	Minimum.	Average.		Maximum.	Minimum.	Average.
Elongation in 8 in.	13	31.6	23.10	27.00	8	29.40	20.70	24.00
Rupture-work......	..	1 460	1 208	1 358	..	2 216	1 203	1 821

These tests, made in 1900, indicate the same phenomena as those made in 1907. In this same paper are tests on three grades of steel castings (coupons from locomotive driving-wheel centers). See Table 64. These three tests would serve to indicate that the harder metal, with less elongation, may take a higher drop of a hammer for rupture than a softer metal of greater ductility.

TABLE 63.—IMPACT RUPTURE-WORK OF VARIOUS MATERIALS.
TENSION IMPACT WITH SINGLE BLOW.

Material.	Diameter, in inches.	Gauge length, in inches.	Impact. Elongation. Percentage.	Impact. Rupture-work, in foot-pounds per cubic inch.	Static. Tensile strength, in pounds per square inch.
Soft steel...	0.50	8	27.00	1 358	68 000
Boiler plate...............	1 by 0.5 (rectangle)	8	34.40	1 855	60 000
Soft steel castings.........	0.50	2	33.00	2 315	62 000
Nickel steel	0.50	8	24.00	1 821	85 000
Steel wire..................	0.16	108	0.70	186	115 000
Steel wire annealed........	0.16	108	9.80	772	83 000
Steel wire annealed........	0.30	108	13.60	765	71 800
Steel wire..................	0.26	108	5.10	556	109 000

*The writer has remeasured the drum records for these impact tests, and finds a correction to be made in the values of the rupture-work in the case of the nickel steel. The correct values are those above.

† *Proceedings*, Am. Soc. for Testing Materials, Vol. IV, 1904.

Mr. Hatt.

This is not always the case, however, as may be seen from Table 63, which is arranged from the publication referred to.

The elongation quoted is that remaining in the broken test bars after the impact test. At the exact time of rupture the elongation is undoubtedly greater by the amount of the subsequent elastic recoil, which latter is no doubt a function of the hardness of the steel.

TABLE 64.—TENSION TESTS IN IMPACT ON STEEL CASTINGS.

Character of fracture.	Kind of test.	Elongation.	Rupture-work.	Contraction.
Silky	Impact.	30.0	2 160	40.0
Flaky	Impact.	19.0	900	18.8
Fine bright	Impact.	22.6	2 811	23.2

Mr. Ostrup.

JOHN C. OSTRUP, M. AM. SOC. C. E. (by letter).—This paper is certainly very exhaustive in its scope, and it is no wonder that so much time was consumed in its preparation, and in making the numerous tests and calculations involved. The information and deductions resulting therefrom are well nigh incalculable in value to the profession in general and to the bridge engineer in particular.

Without any outside assistance, it is manifestly impossible for many individual engineers to carry on costly experiments on such a broad scale, and it is more than doubtful whether any steel manufacturer would care to do so.

It is also improbable that the steel producer will look upon the author's conclusions with a kindly eye; that is, commercially speaking, it is in the interest of the manufacturers to oppose any innovation which will require a refitting, either partially or totally, of his furnaces, his shops, tools, etc., without the full assurance of a commensurate increase, and consequent profit, in the use of the new alloy. This again is doubtful, for some time at least, for the author has shown that superiority in economy does not become a decided factor except for the longer spans, or when the price of carbon steel is high.

However, the work has been completed, the profession has received the benefit, and the writer thinks that too much commendation cannot be extended to the author or to his associates in this undertaking.

Taking up various points in the paper, the first refers to the impact tests made to determine resilience.

The author here correctly states, in regard to this method: "* * * the total amount of abuse given to the metal to be a measure of its toughness and not of its resilience."

This is quite true, inasmuch as an impact test, no matter how made, can only give comparative ideas as to the toughness and ductility of various metals, and not their exact resilience.

The resilience of a structural material is determined otherwise, viz., by its ability to absorb potential energy, under a load or stress, and to restore the same when the cause has been removed. In other words, below the elastic limit, resilience is a measure of ability in materials to perform work. Mr. Ostrup.

When such load or stress is being gradually applied, the resilience, per unit volume, is expressed as follows:

$$\left\{\begin{array}{l}\text{Direct tension, or}\\ \text{direct compression}\end{array}\right\} \qquad \text{Resilience} \ldots = \frac{S^2}{2\,E}.$$

For a beam of rectangular cross-section, such as was used in the experiments, this formula reduces to:

$$\text{(Bending)} \qquad \text{Resilience} \ldots = \frac{S_1^{\,2}}{18\,E}$$

where S = Stress per square inch, at the elastic limit, uniformly distributed over the entire area;

S_1 = Stress per square inch, at the elastic limit, in the extreme fibers;

E = Modulus of elasticity.

Neither S nor S_1 can be ascertained with any degree of accuracy, except where a quiescent load is being gradually applied.

In a drop test, assume a weight, P, falling through a height, h; then the total work $= Ph$. Part of this energy is absorbed by the supports, part by the polar moment of inertia of the beam, and the remainder goes to perform potential work. How great that remainder is, it is not possible to determine with accuracy.

On the basis of the above equations, and using values specified in "De Pontibus," and also in this paper, a better comparison is obtained between the elastic resiliences of carbon steel and nickel steel, viz.,

$$\text{High-carbon steel} = \frac{40\,000^2}{2 \times 29\,000\,000} = 27.66 \text{ in-lb. per cu. in.}$$

$$\text{Low-nickel steel} = \frac{60\,000^2}{2 \times 30\,000\,000} = 60.00 \text{ in-lb. per cu. in.}$$

$$\text{High-nickel steel} = \frac{65\,000^2}{2 \times 30\,000\,000} = 70.42 \text{ in-lb. per cu. in.}$$

Or, calling the resilience for carbon steel 100%, we have:

High-carbon steel..............................	100%
Low-nickel steel..............................	216%
High-nickel steel..............................	254%

These percentages would indicate correctly the comparative resiliences were it not for the facts, as mentioned hereafter, which somewhat modify the same.

Mr. Ostrup. When thus modified, it will be found that the theoretical comparisons will approximate closely to those found in actual tests by Professor Hatt, as referred to by the author.

Is it not quite true that the above comparative values, or in fact any exact comparison, will only obtain where perfect specimens—*i. e.*, specimens uniform in cross-section throughout—are being tested. By the introduction into the tests of deformed or notched bars an element causing great uncertainty has been added.

Compare, for instance, the flexural resistance of two bars, one of which is of somewhat larger cross-section than the other, but has been notched so that its section modulus (but not its moment of inertia) through the notched portion equals that of the smaller and uniform bar.

According to theory, the two bars ought to carry the same load, but this is not so. The effect of the notchings, by contracting the area of the cross-section, is to set up severe secondary stresses, their magnitude being dependent on the relative size and shape of the notches with reference to the un-notched cross-section.

It has been the writer's experience that these secondary stresses reduce the ordinary strength at the elastic limit by from 20 to 40%, and at the ultimate limit slightly more.

While this gives an idea as to the reduction in the static strength of a notched beam, it shows by no means the reduction in its total resilience. This reduction follows an entirely different law, and the effect of this law is to concentrate a large part of the entire work (done upon the beam) on the weak section.

This law is expressed as follows, when considering an elementary area:

$$d\,w = \frac{M\,d\,\beta}{2}$$

where w = Internal work done,
M = Bending moment,
β = Angle after bending between planes which were parallel before bending.

To those who have broken notched specimens, whether in the testing machine or across their knees, it is a matter of knowledge that the angle, β, will be far greater at the point where the cross-section is reduced, hence the work done at that point will increase and the total resilience of the section decrease correspondingly.

That the author was aware of these facts, he indicated by finally intending to use plain bars, and it is the writer's opinion that all tests should be made on perfect specimens and, furthermore, that the "impact" test of any kind of steel be entirely dispensed with.

The results obtained by the corrosion tests, and the lesson these contain, are certainly very instructive, if not somewhat ludicrous.

Thus, for instance, in the acid test the author's high-nickel steel lost 94% in 160 days, whereas that tested by The Osborn Engineering Company lost apparently in the same space of time only about 2 per cent. Mr. Ostrup.

This, however, is not mentioned as a reflection upon the tests themselves, but rather upon the fact that, so far as the writer knows, the profession has no standard specifications covering the method of performing either corrosion tests, or many other equally important tests, upon structural materials.

It has been the writer's experience, in this matter, that nearly every testing bureau or testing laboratory uses a method of its own.

In his specifications for nickel steel, the author advocates its use for the floor systems of bridges, saying "* * * the floor system shall preferably be of nickel steel." To this the writer agrees heartily, inasmuch as there should result a considerable saving in the web by its use.

The economical depth of a stringer, or any plate girder, being dependent upon the allowable unit stress in the flanges, a much shallower depth with nickel steel could be used. Then again the deeper the web (where no intermediate stiffeners are used) the thicker the web must be, in order to satisfy the requirement for a reasonable ratio between unsupported depth and thickness.

In other words, with an allowable ratio of 1:60, a web plate 40 in. deep, using 6 by 6-in. angles, would require a thickness of ½ in., whereas with nickel steel, for the same case, the depth of the web would only be 31 in., and its consequent thickness ⅜ in. Since there is nearly always a great surplus of metal in the web, the waste would be less when using nickel steel.

In his comparisons of costs between carbon steel and nickel steel, with reference to plate girders, it would be interesting to know whether or not the author actually used the reduced economical depths allowed by the use of the stronger alloy.

In many standard specifications giving working stresses for members under a direct stress, no mention or provision is made for stresses due to bending from their own weight, or for other secondary causes. This omission is more general where tension members are concerned. That such bending stresses are by no means negligible can be readily seen by reference to a recent case, where the sectional area of some members ranged between 700 and 800 sq. in., and the lengths between 50 and 60 ft.

In "De Pontibus" the author provides for such bending stresses, and whenever done, the use of nickel steel again will show an advantage, inasmuch as these stresses will be considerably reduced.

The author does not specifically mention suspension bridges, where he demonstrates the economy in the use of nickel steel and its superiority for long spans, but such bridges should undoubtedly be included.

Mr. Ostrup. The case is nearly parallel to that of cantilever bridges, since its dead load is great compared to the carrying capacity, and any reduction in the dead load is also here of the greatest importance.

Furthermore, the use of nickel steel in tall office buildings should be considered in the near future. Some of these buildings in New York City contain from 25 000 to 30 000 tons of steelwork, and, whereas the plate girders or beams might show little or no economy in the use of nickel steel, the writer thinks it highly probable that the columns would show considerable economy in weight and cost, and also advantages in a reduction of the space occupied.

The author states as his opinion that, when used in large quantities, the price of nickel would probably be reduced from 30 cents to 25 cents per pound. Is not that contrary to the laws of supply and demand? Would not the use of nickel steel, or nickel, in large quantities equally well cause an advance in its price?

Considering in its entirety the whole question of the adoption for bridges—or more broadly speaking for structural purposes—of nickel steel, the writer cannot add his voice too strongly to those who are in favor of an alloy which shows so many patent advantages.

There will undoubtedly be some who, for one reason or another, or for no reason at all, will protest against its introduction. Many of our professional forefathers did the same when Bessemer steel was first spoken of, claiming that they had no confidence in its strength or suitability for structural purposes.

Mr. Fidler. T. Claxton Fidler, Esq., (by letter).—This paper conveys information which is of much value and interest, and it goes so thoroughly into the economic advantages of the new material that very little can be said on that head.

The calculations of steel quantities for various spans are very complete, and, although the methods of calculation are not given, the writer cannot doubt but that the special conditions which must affect the design of all compression members, and also those which arise from the undetermined value of the dead load in long spans, have been taken into account. It is only on these points that he ventures to add a few words.

The experiments seem to justify a safe working stress, f_n, for nickel steel, which is nearly $\frac{5}{3}$ the working stress, f_c, that would be adopted for carbon steel; and this ratio of 5 to 3 seems to be applied to eye-bars and also to columns of the ordinary proportions of length, l, to radius of gyration, r. It follows, apparently, that, with the substitution of nickel steel for carbon steel, the sectional area of an eye-bar might be reduced to about $\frac{3}{5}$, making $A_n = \frac{3}{5} A_c$. In the

Mr. Fidler.

case of a column carrying any given load, however, it would not be found practical to effect the same reduction, because it would be difficult to preserve, in both materials, the same ratio of l to r.

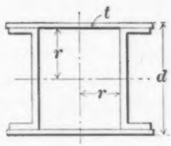


Fig. 76.

Let us suppose, for example, that, in a bridge of given outline and dimensions, we have to design a column of given length, l, to carry a given load; and, first, using carbon steel, we have fixed upon the best diameter, d, radius of gyration, r, and thickness of plate, t, giving a sectional area, A_c, proportional to the product $r\ t$. Then if we proceed to substitute nickel steel for carbon steel and keep to the same radius, r, we should have to reduce the thickness to $t_n = \frac{3}{5}\ t_c$, and the column would fail by secondary buckling or a crumpling of the thin plate which was already thin enough.

On the other hand, if we keep to the same ratio of r to t, in order to avoid this weakness, we should have to reduce the radius to something like $r_n = r_c\sqrt{0.60} = 0.8\ r_c$, or thereabout; and, when the best dimensions are found, the ratio $\frac{l}{r}$ will be greater than before, and the area A_n greater than $0.6\ A_c$.

So far as the compression members are concerned, the economic advantages of the stronger material are only fully realized when the load is great in comparison with the length, l, or, in other words, when there is no liability to buckling; and the advantage is gradually lost as the load becomes less in comparison with the length of the strut. In bridges of the longest span the columns will be of great length, and will be specially liable to buckling; and here we should expect to find, at the same time, that the dead load would be far less in a bridge of nickel steel than in one of carbon steel. The economic advantage of the stronger material, as applied to these long and slender struts, would be comparatively small, just as it has been in the substitution of carbon steel for wrought iron; and, though it could not disappear altogether, it would be less than the advantage of 3 to 5 in the weight of steel required.

On the other hand there is no such limitation in respect to the tension members, and if eye-bars of nickel steel are found to fulfill all the practical requirements, there seems to be no reason why they should not realize the economic advantage to the full. In suspension bridges we should get this advantage throughout the main superstructure; in girders or cantilevers we should get it in the tension members, which form one-half of the structure, while in arches the advantage would be still less.

The writer has long believed that a suspension bridge of rigid

Mr. Fidler. construction would efficiently fulfill all conditions of railway traffic, while realizing other important advantages (a view which was supported on one occasion by Lord Kelvin), and, for spans of 1 500 ft. and upward, it would possess (in many locations) a distinct advantage over the cantilever in point of economy as well as in its rigidity under traffic, its unfailing stability, and its safety under wind pressure.

In any case it is evident that the economic advantage of nickel steel, whatever it may be for small spans, will be enormously greater for bridges of very long span where the weight of the steel itself forms the chief part of the load to be carried. The curves of the author's diagrams show very clearly how the weight of steel, w, per linear foot increases rapidly with the span, L, rising toward infinity as L approaches the "limiting span," $\mathcal{L}$, for any given type of structure.

In a discussion of this matter in "Bridge Construction," the writer has given the formula for weight of main superstructure per foot,

$$w = \frac{L p \Sigma \gamma \mu' + L q \Sigma \gamma \mu''}{1 - L \Sigma \gamma \mu}$$

while the total weight of the structure is $w + p$.

Here the wind metal is reckoned separately and included as part of the platform load, p, and therefore part of the load carried by the main superstructure.

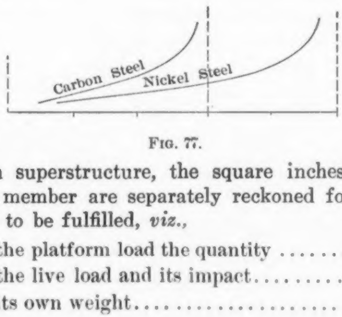


FIG. 77.

For the main superstructure, the square inches of steel section required in each member are separately reckoned for the three functions which have to be fulfilled, *viz.*,

For carrying the platform load the quantity .... $L p \Sigma \gamma \mu'$
For carrying the live load and its impact............ $L q \Sigma \gamma \mu''$
For carrying its own weight...................... $L w \Sigma \gamma \mu$

The formula is applicable to independent girders, arches, or suspension bridges, but, as it stands, not to cantilever bridges; and for a given type of design the quantities, μ, μ', and μ'', are independent of the material used. The limiting span, $\mathcal{L}$, is obviously reached when $1 - L \Sigma \gamma \mu = 0$, or when $L \Sigma \gamma \mu = 1$, so that $\mathcal{L}$ is inversely proportional to the specific weight, γ, per ton of stress, or directly proportional to the working stress, f. For each type of structure in nickel steel, $\mathcal{L}_n = \mathcal{L}_c \frac{f_n}{f_c}$; and for the suspension bridge we should expect $\mathcal{L}_n = \frac{5}{3} \mathcal{L}_c$.

The curve representing the relation of w to L is easily traced; and its general form agrees with the curves of the author's diagrams, as shown in Fig. 77. Mr. Fidler.

ROBERT E. JOHNSTON, ESQ. (by letter).—The writer congratulates the author on the results he has obtained in his investigation of the physical characteristics of nickel steel as compared with carbon steel. Mr. Johnston.

After a careful perusal of the paper, the writer is of the opinion that, with the proportions of nickel determined by the experiments, nickel steel has been proved to be a material which can be safely relied on in the construction of bridges of large span, and that considerable economy can be obtained by its adoption as compared with carbon steel; but it is a question whether the difference in cost will justify its adoption in bridges of small span, because, for structural reasons, full advantage cannot be taken of the difference in the strength of the two materials, for instance, in the webs of plate girders.

The difference in the strength of nickel steel for rivets as compared with carbon steel, requires that, for the latter material, additional metal must be provided in order to compensate for the area removed from the plate, owing to the increased diameter to be given to the nickel-steel rivets, as stated in the paper.*

Another great advantage of nickel steel over carbon steel is that the former is affected to a less degree by the atmosphere and the fumes from locomotives. This is of great importance in the maintenance of large bridges, for it necessitates a considerable outlay in scaffolding for painting purposes, but, to a certain extent, this is discounted when the two metals are used in the same structure.

The photographs of the columns tested (Plate XIX) would appear to indicate that the unsupported wing of the angles should be made thicker than the other wing, which would have the effect of increasing the radius of gyration, and thus add to the strength of the column. It would add to the interest of the experiments if they included cylindrical columns, in order to determine their strength and also to ascertain the ratio of the thickness to the diameter.

ALBERT LUCIUS, M. AM. SOC. C. E. (by letter).—The writer would most assuredly build long-span bridges of nickel steel. The disclosures in reference to the Quebec Bridge and the Blackwell's Island Bridge would not leave a moment's hesitation in his mind. With equal assurance, he would build short-span bridges of soft steel, and would not even be influenced by the possibility that nickel-steel bridges might be as cheap or cheaper, simply on grounds of greater perfection. The writer is not sure where he would draw the line, but, in a general way, he would draw it when dead-load stresses exceeded live-load stresses, and would then commence to study the matter in detail. Mr. Lucius.

* Page 116.

Mr. Lucius.

Dr. Waddell's paper is a complete-enough proof that nickel steel is fully available. The writer would prefer heavy bridges of soft steel to light bridges of nickel steel for railroad purposes, because he would consider them steadier under heavy loads and high speeds. When the practical limitations of manufacturing bridges are reached he would use nickel steel to help out, and would probably make the entire construction of nickel steel, and not only a few members. The writer considers this paper to be highly valuable, and believes that the thanks of the Profession are due to the author.

Mr. Lindenthal.

G. Lindenthal, M. Am. Soc. C. E. (by letter).—From his experiments, the one conclusion which Mr. Waddell evidently desires to emphasize most is that nickel steel is economical, as compared with the usual structural carbon steel, for nearly all kinds of steel structures. Mr. Waddell's reasons for such a claim are ample and weighty, but not always applicable. It is also doubtful if the economy shown in his tables could be obtained if nickel were extensively used. A large demand for it would probably raise its price. Nickel is not a very abundant metal, nor is it very cheaply produced.

The use of high steels, nickel steel being one of them, in bridge construction, is justified only in very long spans, meaning that length of span in which the dead load is considerably greater than the live load. The average stress in bridge members from live load is then smaller than that from dead load, and that keeps the deflections of the structure within moderate limits.

In shorter spans, when the stresses from live load exceed those from dead load, the structure becomes too springy, vibrations become greater, and the effect of impact, of which we know so little, becomes very pronounced.

We recognize the relation between train and bridge to be that of hammer and anvil. The anvil would not last long if it were much lighter than the hammer. To use high steel (including nickel steel) in a light structure, for the sake of first low cost, is not true economy.

Although nickel steel was proposed for bridge construction long ago,* the writer was the first, he believes, to introduce it practically in the form of forged eye-bars, namely those for the Blackwell's Island Cantilever Bridge over the East River in New York. The design for this bridge originated with the writer (in 1903), but he resigned from the charge of this work about one month after making the contract for it. He had no connection with, and is not responsible for, the changes subsequently made, which resulted in an inferior structure. Among the changes attempted was the substitution of hard (high-carbon) steel eye-bars for those of nickel steel; but the tests with them were failures, and so the eye-bars of nickel steel were retained, and these are the bars described in Mr. Waddell's paper.

* "Suspension Bridges—A Study," by George S. Morison, *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 359.

Mr. Lindenthal.

To avoid the great loss of strength in forged eye-bars through annealing, it would be more economical to make large eye-bars by cutting them from plates having the width of the head, as was done for the eye-bars of carbon steel in the new Budapest Suspension Bridge.

According to Mr. Waddell's tables, it would seem that cantilever bridges are the only ones worth considering for long spans; but that is not true, even from the purely commercial point of view, as there are other systems, as arches and continuous girders, which for long spans are more economical and more rigid. The cantilever systems mentioned in his tables are, of all types, just the ones in which nickel steel would increase their already sufficiently great defect, that is, cumulative vibration and lack of rigidity.

Nickel steel will be found to be most appropriate and economical in the suspension type of bridge. The writer, in 1903, proposed that material for the eye-bar chains of the Manhattan Bridge over the East River, with spans of 725, 1 470, and 725 ft.*

Had a competition of the nickel-steel chain design there described, viz., steel-wire cables design, not been suppressed by interests adverse to the public good, there is no doubt that the nickel-steel eye-bar bridge would have been found very much cheaper, besides being better in every other way.

The wrong perception which is still prevailing among engineers in regard to the nature of suspension bridges of long span, and, when properly designed, their eminent safety and suitability for heavy and fast railroad trains, may be explained perhaps from the fact that such bridges can be built in only a very few localities, and therefore have been insufficiently studied. In the writer's opinion, nickel steel and other high steels are the most suitable materials for them.

Mr. Prichard.

HENRY S. PRICHARD, M. AM. SOC. C. E. (by letter).—Nickel-steel eye-bars have been manufactured and used to an extent which justifies their classification as an article of commerce when ordered in sufficient quantity, but so meager has been the use of nickel steel for the riveted members of structures that it has hardly reached the experimental stage; rather it is a matter of academic discussion.

It is possible, of course, to draw some conclusions from tests such as the author describes, but the most that can reasonably be expected from them is that they will help to determine whether or not it is worth while to try the experiment of riveted nickel-steel structural work. Even if preliminary investigation is favorable, nothing but

* "General Methods for the Calculation of Statically Indeterminate Bridges, as used in the Check Calculations of Designs for the Manhattan Bridge and the Blackwell's Island Bridge, New York," by Frank H. Cilley, *Transactions*, Am. Soc. C. E., Vol. LIII, p. 413.

"A Rational Form of Stiffened Suspension Bridge," by Gustav Lindenthal, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LV, p. 1.

"Theory and Formulas for the Analytical Computation of a Three-Span Suspension Bridge with Braced Cable," by Leon Moisseiff, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LV, p. 94.

Mr. Prichard. actual experience can show conclusively the suitability of the material and the cost of fabrication.

The suitability of the material has to be gauged, not only by the ordinary treatment it receives in fabrication, erection, and service in structures, but, in addition, by the abuse it can stand and to which, in spite of reasonable care, it is more or less subject, both before and after the structures are completed.

Existing data are not sufficient to admit of a close reliable estimate of the relative cost of nickel and ordinary steel riveted work, but the writer regards the author's estimate as too favorable to nickel steel.

The position of engineers with regard to the use of nickel steel is similar to that of the girl whose mother expected her to learn to swim without going near the water; they should not use nickel steel until they have had experience with it, and they cannot get the experience until they use it.

Those who desire to be pioneers in the use of nickel or other special steels, if the preliminary investigations are sufficiently favorable, should consider other matters in addition to the results of experiments and preliminary estimates of cost. For the ordinary run of structures, if structural steel of a special quality is desired, the quantity is comparatively so small that it is difficult, even at an increased cost, to obtain it. When it is possible to do so, it would doubtless give greater security and longer life to such structures to put the increase in cost into more metal of standard quality. There is great virtue in body, and many structures could have more with advantage.

Especial caution is necessary in using for ordinary structures unit stresses greater than would be safe in ordinary carbon structural steel. Even if the engineer is satisfied that he can rely on the superior strength of some special kind, he must reckon with the fact that such steel cannot be distinguished from the ordinary variety by simple observation, and that, unless the mills are making and the manufacturer using the special steel exclusively, there is a likelihood that some steel of ordinary quality will get mixed with it, in spite of the best intentions and reasonable care. If he succeeds in getting only the special steel, there is danger that in course of time the structure will come under the supervision of other engineers who will not know the quality of the steel, and who, in judging as to safety, will be obliged to assume that it is no better than ordinary.

For bridges of very long span, the dead weight is such a large proportion of the entire load that the comparative lightness obtained by the use of steel of great strength is a decided advantage, especially for tension members. Members in compression, in addition to resisting the effort of the load to crush them, have to resist its tendency to buckle and wrinkle them, and the resistance to these tendencies is about the same for steel of all grades.

For bridges of very long span it is possible, and for the eye-bars of such spans it is probable, that the advantage of steel of great strength may offset the disadvantage of increased cost. There is, however, great need for caution in regard to riveted members. Mr. Prichard.

For eye-bars there is, fortunately, a precedent in the Blackwell's Island Bridge, the full-sized tests of which are given in Table 60, which shows the results of 125 full-sized tests. The percentage of nickel varied from 3.22 to 3.76, with an average, according to the author, of 3.39. The percentage of carbon varied from 0.32 to 0.46, with an average, according to the author, of 0.39. The ultimate tensile strength per square inch varied from 80 360 lb. (16 by $2\frac{1}{8}$-in. bar) to 98 980 lb. (16 by $2\frac{1}{8}$-in. bar), and the recorded elastic limit from 39 960 lb. (16 by $2\frac{1}{16}$-in. bar) to 60 570 lb. (16 by $1\frac{15}{16}$-in. bar). The percentage of elongation in 18 ft., omitting bars that broke in the head, varied from 8.44 to 18.44. In 27 of the 125 cases the recorded elastic limit is less than 55% of the ultimate strength, and in three cases it is less than 50 per cent. Owing to the large number of tests, the character of the plant which manufactured the bars and made the tests, and the meagerness of other available data, they should carry great weight with engineers in posting themselves as to what it is practicable to manufacture, and in avoiding what it is useless to specify.

The author tested eight nickel-steel eye-bars which he states were almost identical in composition with those for the Blackwell's Island Bridge. The ultimate tensile strength per square inch varied from 88 900 lb. (8 by 2-in. bar) to 103 200 lb. (6 by 1-in. bar), and the recorded elastic limit from 48 300 to 58 700 lb. per sq. in. The percentage of elongation in 10 ft. varied from 10.6 to 14.5. In four of the eight cases the elastic limit was less than 55% of the ultimate strength, and in one case it was 51 per cent. Two of these four cases were 6 by 1-in. bars, a size in which the best results are attainable.

In addition to the tests cited, the author tested two eye-bars fabricated from $\frac{3}{4}$-in. plates, with planed edges, of 4.25% nickel and 0.46% carbon. The results of these tests, naming number one first in each case, were: Ultimate intensity, 105 900 lb. and 102 300 lb.; elastic limit, "Lost" and "Uncertain"; percentage of elongation in 10 ft., 7.4 and 6.8.

The author's specifications for eye-bars call for: Nickel, 4 to 4.5%, and carbon 0.40 to 0.50%; and require full-sized tests to show an ultimate tensile strength per square inch varying from 90 000 lb., for $2\frac{1}{2}$-in. or greater, to 105 000 lb. for 1-in. They require an elongation of not less than 10% in a gauged length of 10 ft., and an elastic limit of not less than 55% of the ultimate strength of the bar. These specifications are much higher than are warranted by the tests cited. With two exceptions, the tests have no direct bearing, as the steel had lower percentages of nickel and carbon, but, inferentially, they do not

Mr. Prichard. support, but are against, the author's specifications. Of eye-bars of about the quality specified, the author cites only two full-sized tests, neither of which is up to the specifications. In discussing these two tests, it should be remembered that the edges of the bars were planed, that the manganese was less than the specifications require, and that the author is of the opinion that rolled edges and manganese as per specifications would have produced better results. Until he gets the desired results, however, and in sufficient quantity to show that it is practicable to rely on them in practice, his specification for eye-bars is not warranted.

The author's specification of 55% for the elastic limit is in harmony with his statement on page 139, under "Full-Sized Eye-Bars," that "The elastic limit in nickel steel never falls below 55% of the ultimate strength," but it does not agree with the tests of the eye-bars for the Blackwell's Island Bridge, nor with his own full-sized eye-bar tests, for half of which (four out of eight) the recorded elastic limit is less than 55% of the ultimate strength, with a minimum of 51 per cent. It should be noted here that the specifications require that: "the elastic limit shall not be less than 55% of the ultimate strength of the bar," instead of limiting it to 55% of the ultimate strength required. The difference can be illustrated by reference to two eye-bars 2 in. thick, the first with an ultimate of 95 000 lb. (the amount required) and an elastic limit of 52 250 lb., and the second (an actual bar) with an ultimate of 95 880 lb. and an elastic limit of 52 360 lb. The first bar conforms to the author's specifications, but the second bar (of the same size) does not, although it has a higher elastic limit, because this limit is less than 55% of the ultimate strength of the bar.

Structural steel which has not previously been subjected to an external load usually displays slight imperfections in elasticity, and acquires a slight permanent set before, and sometimes long before, there is any considerable loss in elasticity. Eventually, under a gradually increasing load, if the steel is moderately soft, the yield point is reached where the low rate of change, characteristic of steel strained within its elastic limit, suddenly changes to the high rate which results from the flow of the metal after the destruction of its elastic properties. As the percentage of carbon is increased, the abrupt change in the rate of extension becomes less marked and finally disappears, so that there is for such cases no sharply defined yield point. The attempts to define it clearly for hard steel are arbitrary, and when rigidly adhered to may be unjust and unwise.

The elasticity of steel within certain limits is made nearly, perhaps quite, perfect by a second or subsequent application of the load after a suitable interval of rest, but the material has to take some permanent set in order to reach this condition. As a basis for designing steel structures, the critical question with regard to strength is: "What

is the greatest permanent or indefinitely repeated load which structural steel will sustain without undue deformation?" It would be very satisfactory to have this question very closely settled in each case by some direct and simple test. Unfortunately, this is not possible. Judging from the tests and experiments which have been made, it is not wise to count on an elastic limit for eye-bars of more than 50% of the ultimate strength, or to expect that there will not be slight imperfections in elasticity below this limit on the first application of the load. Mr. Prichard.

The author quotes from a paper by the English authors, Messrs. Carpenter, Hadfield, and Longmuir, to the effect that steel containing from 0.40 to 0.50% of carbon and 4½% of nickel has lost none of its toughness or ductility, but that a steel containing 5% of nickel has lost both to a serious extent. He is of the opinion that between these two percentages there is probably a well-defined point of demarcation which can only be determined by further experiments. Would it not be well to wait for more information on this point before specifying eye-bar steel with nickel from 4 to 4½ per cent.?

The author's specifications allow 66⅔% greater tension in nickel-steel eye-bars than in those of ordinary carbon steel. In the writer's judgment, this is not warranted by experiments, and is too great an increase, especially when applied to unit stresses as high as those quoted from "De Pontibus." The calculated stresses are only approximations, and are not always close or complete; besides which, the likelihood of defects, the possibility of blunders, the certainty of deterioration, and the chance of accidents and unforeseen contingencies, make it unwise, to say the least, to narrow the margin of safety.

The matter of annealing eye-bars, as the author states, is worthy of further investigation. A complete understanding of the phenomena involves a knowledge of the exact nature of steel and the reasons for its properties. The nature of steel is, at present, largely a matter of conjecture, and therefore, it is not yet possible to make a satisfactory explanation of annealing. The prominent conditions and results in any given case can be observed, and, of course, the inference can be drawn that similar conditions will produce similar results; the results, however, can be decidedly affected by many things, some of which are quite subtle, both before and after the metal reaches the annealing furnace.

Eye-bar manufacturers do, of course, make earnest study of the phenomena of annealing, and reach well-supported conclusions. In the case of the nickel-steel eye-bars for the Blackwell's Island Bridge, the American Bridge Company made more than 250 full-sized tests. A very large proportion of these were made in advance of commercial manufacture, to study the annealing processes and temperatures necessary to produce satisfactory results. On page 137 the author seems

Mr. Prichard. to question the care with which the eye-bars for this bridge were annealed. As he states, he was not furnished with information concerning the annealing, nor was he furnished with records of the experimental tests which, from the nature of the case, were really much more instructive than the tests on which the bars were accepted, and which he reported in Appendix B.

The range of experimental tests on annealing made by the American Bridge Company was very much wider than that contemplated by the author, as outlined in the last paragraph of page 134 in variety of sizes, number of pieces of full length, number of pieces annealed at one time, etc. Temperatures, also, were varied within comparatively wide limits. These experimental tests established the necessity of treating each bar in accordance with its own particular chemical and physical characteristics; and this experience was applied with great care to the commercial product in a well-equipped and thoroughly controlled furnace.

The opportunities of eye-bar manufacturers for observation are so great that their advice in the various cases which arise should be very valuable.

Experience admits of some general conclusions regarding the annealing of eye-bars, but it is not wise, in the present state of knowledge, to formulate the process rigidly.

Eye-bars are not annealed for the purpose of improving the quality of the material as it comes from the rolls, but simply to offset the undesirable results of forging the heads. In annealing eye-bars, the metal is softened, and some of the good effect of rolling is taken away. The end which should be sought is to offset the possible ill effects of forging the heads, with as little change as possible in the condition of the metal as it comes from the rolls, and this end should be kept in mind when applying the well-known fact that eye-bars which cool slowly will be softer than those which cool more rapidly.

The thanks of the Engineering Profession are due to the author for the information which he has contributed in this paper, and he should be commended for the earnest efforts and generous expenditure which made this contribution possible. In endeavoring to establish closely, by tests and analysis, in advance of experience, the engineering worth and economic value of nickel steel as a structural material, his aspirations were very high, and he set for himself a task which few engineers would care to attempt. While, owing to practically insurmountable difficulties, he has not accomplished all he desired, he has succeeded in increasing greatly the general knowledge regarding this material.

Mr. Le Chatelier. HENRY LE CHATELIER, ESQ. (by letter).—The advantage of using steel of high resistance in the construction of long-span bridges is too evident to require any emphasis. Attention has already been called

Mr. Le Chatelier.

to this subject by Mr. Louis Le Chatelier.* After a series of breaking tests on beams made of silicon steel, having an ultimate strength of 101 000 lb. per sq. in., he first used this steel in an overhead crane girder, with a reduction of 25% in the weight of metal, and then in the derrick of an erection traveler where the weight reduction was still greater.

Unquestionably, the same advantages can be obtained with nickel steel as with silicon steel; the choice of one or the other of these metals will be essentially a question of net cost. It would be of great interest to test these special metals systematically, and on a large scale, in heavy engineering constructions. Even if the cost should be greater than Mr. Waddell actually admits, it would still be advantageous to make these trials, at least by such agencies as railroad companies, or State departments, which often have to renew constructions of this kind. Even should the cost of these trials be excessive, they would still, in the end, result in structural advantages by which the above-named agencies would be the first to benefit.

It is doubtful, in the meantime, whether the use of these special steels will become general, unless there is a revolution in the net cost of the raw materials. It is very probable that, after a short-lived use of special steels, engineers will return to the ordinary carbon steels, but with the latter very much harder than they are now, just as was the case with the construction of torpedo-boats.

On the advice of the French constructor Normand, there were adopted, some twelve years ago, for the hulls of torpedo-boats, plates made of nickel steel containing 3½% nickel, and a proportionally high amount of carbon, altogether similar to the steel treated by Mr. Waddell; but very soon the steel manufacturers offered carbon-steel plates possessing the same resisting qualities, at a less cost, and it will certainly be the same for large metallic structures. Neither nickel nor silicon adds to the steel any special qualities of resistance, but these qualities are obtained with less care in manufacture than is required to produce a steel that is high in carbon and of correspondingly high strength, but without the fault of brittleness. It is much more difficult, and requires much more care in manufacture, to obtain these results with steel containing carbon only, but it is not impossible. The use of nickel steel must only be considered as a step, certainly a very useful one, and perhaps indispensable, in the advance toward the final use of hard carbon steel.

In the desire to pass directly from the present soft steel to hard steel, there is danger of errors of application which may be of such a nature as to retard for a long time the definite use of steel of high resistance. The work of Mr. Waddell, therefore, cannot be too highly encouraged.

* *La Revue de Métallurgie*, 1904, p. 646.

Mr. Ross. A. Ross, Esq. (by letter).—Dr. Waddell's paper comes at an opportune time, inasmuch as hitherto information on this subject has consisted of little more than detached paragraphs in the press; and this paper concentrates the past and gives valuable information and experiments for the first time. The subject is ripe for consideration, inasmuch as engineers have been studying for some time the value of nickel steel.

In Great Britain few structures, such as bridges, have been constructed of nickel steel. Any experience the writer has had has been solely in connection with rails, having experimented on two or three occasions with steel containing, in one case, 3.3% of nickel, and, in the other, 1.5 per cent. In both cases the physical results obtained were better than with ordinary steel; but, looking at the matter comparatively, and taking price and wear together, nickel steel did not show to much greater advantage, commercially; in some cases the nickel had not been uniformly distributed, in other words, the metal was not altogether homogeneous, and the writer came to the conclusion that the manufacture was rather more to blame than the analysis.

Undoubtedly, it would be a great advantage in bridge building if one could secure a reliable alloy, which would bear higher stresses than ordinary carbon steel and, consequently, would be of relatively less weight. As far as the writer knows, steel containing a percentage of nickel is the only metal of this kind we can look for.

The writer observes that the nickel steel with which Dr. Waddell experimented was manufactured exclusively by the "basic open-hearth process", which he understands is the process adopted almost everywhere in the United States, and which the author assumes will be that absolutely used in the future; but, in Great Britain, the acid open-hearth process is more generally used than the basic open-hearth. At the same time, nickel-steel rails can be manufactured with facility by the Bessemer process, which is that largely used in Great Britain for steel rails.

With the little experience the writer has had with steel containing a percentage of nickel, he can easily assent to the valuable statement made by the author that for plates and shapes, which form the principal part of bridge construction, any material with a greater percentage of nickel than 3.5 renders the alloy too refractory for the various shop manipulations to which it must be subjected in being manufactured into bridges. This fact, being established, fixes the limit of the percentage of nickel to be used to meet general requirements.

As to carbon: It is pointed out that an addition of carbon might raise materially the ultimate strength and elastic limit of the metal, but this again is limited by the facility of workability in the shop, and clearly it would be more or less dangerous to have two classes of

metal on the ground for the same purpose, viz., those which could go under manipulation in the shops and those which could not, as there would be difficulty in keeping them separate. Mr. Ross.

The author's numerous tests, finally leading up to his conclusions, are very instructive, and comparing these with carbon steel, the safe limit of which, in Great Britain, is taken at 6½ tons, both in compression and tension, and at the standard of 1.00, the comparative results given by nickel steel are as shown in Table 65.

TABLE 65.

	Tension.	Compression.	Bending.	Bearing.	Shear.
Eye-bars	1.67	....	....	....	....
Built members	1.75	....	....	....	....
Net section of flanges	1.71	....	....	....	....
Pins	....	....	1.85	1.73	1.67
Rivets	....	....	....	1.50	1.40
Webs of plate girders	....	....	....	....	1.70
Top chords of railway bridges	....	1.65	....	....	....
Columns, fixed ends	....	1.55	....	....	....
" hinged ends	....	1.48	....	....	....

With the smaller members of a bridge it is difficult, even now, with carbon steel, to limit the sectional area of the bar to its absolute requirements, and it becomes more so with a metal of greater unit strength, such as nickel steel. The author's anticipitation of ordinary county bridges vanishing into thin air might be realized with disastrous effects on small railway bridges.

The corrosion tests that were made do not appear to be quite satisfactory or convincing, and, in the writer's opinion, they do not lead to the conclusion that "nickel steel resists [corrosion] decidedly better than carbon steel," but leave the question open for further experiment and consideration. With equal resistance to loss of weight by corrosion, a bar of nickel steel will deteriorate in strength 50 or 60% faster than a similar bar of carbon steel.

Economical considerations will lead to the use of nickel-steel plates thinner than those of carbon steel. If the thickness is reduced relatively to the strength of the alloy, for example, if a ⅜-in. nickel-steel plate is used instead of a $\frac{9}{16}$-in. carbon-steel plate, then the percentage of the weight of bar lost by corrosion in the nickel steel will be one and one-half times as great as in the carbon steel. This increased loss of weight is equivalent to an increased loss of strength in the nickel steel at least two and one-fourth times as great as in carbon steel. The introduction of nickel steel will involve, therefore, an increase in the present allowance for corrosion.

At prices now current for nickel and for structural steel in Great Britain, the diagrams show that there is no economical reason for the

Mr. Ross. introduction of nickel steel for railway bridges of less than 260 ft. span in the case of double-line, or about 160 ft. span in the case of single-line bridges.

Probably for a long time after the first introduction of this use for nickel steel, bridges of spans much exceeding the above limits would be built more cheaply of carbon steel, unless the price of nickel were to fall very much below its present rate of from £170 to £175 per ton.

For small spans (30 ft.), the costs of nickel-steel, wrought-iron, and cast-iron bridges, compared with carbon-steel bridges, work out as follows:

Carbon steel	1.00
Cast iron	1.07
Wrought iron	1.20
Nickel steel	1.23

In Great Britain plate girders of ordinary carbon steel cost about 3 cents per lb., or £13 16s. 0d. per ton; truss girders cost about 3½ cents per lb., or £16 2s. 0d. per ton. If built of nickel steel, the increased price will not be less than 1.6 cents per lb., or £7 7s. 2d. per ton extra in each case.

This gives a very large increased strength per unit, and a corresponding weight gives a very much less dead weight of metal required in the case of nickel steel as compared with carbon steel, clearly showing that an engineer can afford to pay a greater price per ton for nickel steel than for carbon steel, but to what limit the writer is not prepared to say at present.

No doubt many things are still required to be ascertained, and one feels that the manufacture of steel is always a delicate matter, and the more complex it becomes the more uncertain is the reliability. If chemists and engineers could put their heads together and produce definite conclusions insuring reliability, it would be of the greatest advantage.

The commercial question must also enter largely into the matter, and must be considered in each specific case as to whether, in view of the higher cost of nickel steel, it may not be of advantage, with bridges up to a limited span, to adopt the steel generally used during the past series of years.

No doubt as time goes on and nickel steel becomes better known, facilities for its manufacture will be extended and improved, and it may replace the older carbon steel.

One of the uses to which nickel steel may be applied—that of reinforcing concrete—has not been dealt with by the author. It is a use to which one can imagine it might be successfully applied, and, if data could be obtained as to its freedom or otherwise from corrosion when embedded in concrete, it would be most useful, inasmuch as its superior

strength and comparative lightness render it suitable for such a purpose. Mr. Ross.

The question as to the value of nickel steel is in a large measure a new one awaiting examination and test, and the Engineering Profession at large owes Dr. Waddell a debt of gratitude for the able way in which he has dealt with the subject. The writer trusts that he will continue his investigations.

L. DUMAS, ESQ. (by letter).—The author is to be congratulated on having undertaken and carried out so well the very important studies recorded in this paper. Mr. Dumas.

The introduction of nickel steel into the construction of bridges is one of the most desirable advances, because great opportunities for it will be developed, even though it has not yet been adopted for special structures of small tonnage.

The key of the problem is to strike a balance between the great advantages of nickel steel judiciously chosen and the increase of cost resulting from the introduction of this expensive metal. It seems that Dr. Waddell succeeds in doing this.

In the following observations, the writer will not discuss the experiments on bridges or bridge members, as this is not his particular sphere; but will speak from the standpoint of the metallurgist.

The first question presented is the chemical composition of the steel. The paper notes, on good grounds, the agreement of technical and economical considerations in limiting the nickel contents. They have led him to adopt $3\frac{1}{2}\%$ of nickel for plates and bars. The writer thinks there is no possibility of exceeding this percentage; perhaps it is even too ambitious to introduce nickel in members which should not be annealed.

In effect, nickel steel should be considered as nickel-carbon-manganese steel. The phosphorus and sulphur are injurious elements, and should always be reduced to a minimum, but the carbon and manganese are the constituent elements, active, like the nickel, mainly in increasing the limit of elasticity. One should sum up the actions of these three elements in considering their coefficients, which are far from being the same; the carbon is by far the most active, the manganese comes next, then the nickel. Account should be taken of the effect produced by these additions by considering it as a hardening, analogous to that which results from rolling or cold-hammering; with this difference, however, that this hardening is not superficial, but is produced in the whole mass, as if the molecules of iron were wholly compressed by the foreign elements. It is known that the hardening increases the elastic limit and the resistance to rupture, and diminishes the elongation and the resistance to shock. It is known, also, that it improves the quality of extra mild (very soft) steel, but when it is

Mr. Dumas. too intense it becomes injurious. In the same way, an addition of nickel is never injurious if less than 2%, but may become an advantage.

One might be tempted to substitute some carbon or manganese for nickel because their properties are similar, but this substitution should not be made, partly because, for a like increase in the elastic limit, the nickel diminishes to a less extent the elongation and the resistance to shock.

The proportions: Nickel 3.5, manganese 0.70, carbon 0.38, are probably the highest which should be adopted for bridges, because the dimensions of the parts do not permit of submitting them to be annealed, a treatment which becomes necessary in order to diminish the interior tensions produced by the introduction of these three elements.

The foregoing are the principal thoughts suggested by this paper. It is hoped that the author will pursue his experiments, as they appear to have been conducted along the proper lines.

Mr. Perry. VICTOR PRITTIE PERRY, ESQ. (by letter).—This paper has been read with much pleasure, and its subject cannot fail to interest in the highest degree all railway and bridge engineers, and to a greater extent, perhaps, all practical steel manufacturers.

It would seem to the writer that the necessity for, and utility of, further research are fully demonstrated. Some of the experiments fail to agree, others are not quite conclusive, and there may be a couple of points which have not been mentioned at all. The facts that have been brought to light prove beyond dispute the utility of the subject under discussion.

There appears to be insufficient information collected at present with regard to the composition of the alloy. The work done is most instructive, as far as it goes, and shows apparently that there will be a wide field for the use of nickel steel, but further experiments are required in which carbon, manganese, and nickel are varied, and varied largely. It is impossible to say what results may not be obtained.

Dr. Waddell does not say why it was decided to leave the acid open-hearth steel out of the question. It would be a pity if any recognized method of manufacture of carbon steel should be barred for the production of the new alloy, for, if so, it would probably have a marked effect on the cost of its production.

It appears plain that nickel steel is not as ductile as medium carbon steel, and the effect of cold on the bending tests is very marked. Were any tensile tests made with the alloy at different temperatures? It would almost seem that those were required. Nickel steel is stated by some authorities to be more ductile than medium-carbon steel.

It would also be advisable if a series of experiments were made on the effect of continued vibration on nickel steel, as the results of

such experiments are required in order to fix satisfactorily the working loads for short spans, bridge floors, cross-girders, etc., if it is ever profitable to use the steel in such places. From a practical point of view, one would like to know whether there will be any greater difficulty in straightening members of structures bent in transit in the nickel steel than in the carbon steel; and how would it best be done, that is to say, cold, or after heating. Mr. Perry.

The effect of the acid test would seem to point to the inadvisability of using the steel in a dense manufacturing district like Lancashire; but the result of the smoke test appears to be curious in comparison with the acid test, as it is generally believed that it is the sulphurous acid in the smoke that damages structures.

The result of the torsion test surprised the writer; it would seem to show that nickel steel is not in every respect stronger than medium carbon steel, but perhaps further experiments with different proportions of the three principal ingredients will reverse the result now obtained. The resiliency tests are also inconclusive, and further experiments under this head are required. The fact that three of the four results favor carbon steel would seem to prove that there was something faulty in the methods adopted, and these perhaps could be improved and extended.

It will be necessary to devise some method of fixing definitely and reliably the "yield point" and the "elastic limit," and these two terms should not be confused. The specified method of determining the "elastic limit" would seem, properly speaking, to be more appropriately styled the method of determining the "yield point."

Speed in determining the tensile strength is admirable, if it can be attained, but it is only a secondary consideration, and the more complex an alloy is, and consequently more troublesome to manufacture, with correspondingly greater possibilities of error, the greater necessity for more painstaking and vigorous tests. The more expensive an alloy is, the more time can be spared by the manufacturers, and should be demanded by the purchaser, for careful tests, and such tests should not be proportionately more expensive for the dear alloy than for the cheap one.

Everything in the experiments seems to point to the necessity for specifying very carefully the nature of the tests that will be required, and in such specifications the speed of the testing machine and the thickness of the specimens will require to be carefully detailed.

The experiments on columns are among the most interesting features of the paper, but the results are puzzling. From the usual formulas for the strength of columns, which have again lately been investigated and extended by Dr. Lilley, of Trinity College, Dublin, it has always been accepted that columns and struts of the same length and similar cross-section, differing only in the material of which they

Mr. Perry. are composed, should have their ultimate strengths differing in the same ratio that the compressive strengths of the materials differ. As far as our present knowledge (which is not very great) on the strength of columns goes, the above law holds, but the tests of columns of medium carbon and nickel steel do not seem to bear out this law, because the ratio of ultimate strengths of 10-ft. columns is given as 175, and for 30-ft. columns it is only 147. The ratio of loads producing a permanent set of 0.005 in. varies still more, being 183 for the 10-ft. columns and 245 for the long columns. It is to be hoped that the experiments already made will be largely extended, and it would not be surprising if the ratio given for the failure of the long columns should finally be found to be more in accordance with facts.

No tests except those referring to the "bearing-on-pins" tests appear to have been made in connection with the ultimate compressive strengths of the nickel and carbon steels; such tests would seem to be needed.

The working stresses proposed for nickel steel, if the elastic limit and ultimate strength given are to be accepted, would not be regarded as satisfactory in Great Britain. With a practically new alloy, a far larger factor of safety should be adopted. American practice, in this respect, with regard to medium carbon steel, is so different from that in use for many years in the United Kingdom that the views of a British engineer will hardly find favor, but in view of some recent bridge failures in America, it would seem that the current American practice could be modified with advantage.

The difference between maximum allowable working loads in America and the United Kingdom will render it somewhat troublesome for a British engineer to use the excellent diagrams, which must have given the author a great deal of trouble to prepare, showing the relative weights and costs of nickel- and carbon-steel bridges.

The advisability of mixing nickel and carbon steels ultimately in bridgework, such as in plate girders, making the booms or part of them of nickel and the webs of carbon steel, seems problematical, and Dr. Waddell's remarks on the inadvisability of using nickel-steel rivets with carbon-steel plates would point in the same direction. To the writer it does not seem likely that nickel steel will be used to a great extent for bridges of less than 150 ft. span, as there must be a certain minimum dead load in comparison with the live load, and, as far as the writer's experience goes, he would not care to cut down the minimum weights which can be obtained by careful design with carbon steel in short-span bridges, using the working stresses regarded as suitable in the United Kingdom.

For bridges of 300 ft., or thereabout, no doubt the use of nickel steel would prove economical, and for bridges of the maximum span it will be indispensable.

In conclusion, the greatest praise and most hearty thanks are due the author from the Engineering Profession for this very excellent paper. The writer feels sure that only a few realize the amount of work involved in its preparation. No doubt Dr. Waddell's desire will be attained if the paper leads to further experiments, as there is little doubt it will. Mr. Perry.

W. H. Warren, Esq.* (by letter).—The writer's experiments on nickel steels were published in 1898 and 1902.† Recently, some specimens of this steel have been tested in Charpy's and in Guillery's impact testing machines, using standard notched bars. The material was supplied by the firm of Fried. Krupp, of Essen; its composition is shown in Table 66. Mr. Warren.

TABLE 66.

Reference letter.	Carbon.	Silicon.	Manganese.	Phosphorus.	Sulphur.	Copper.	Nickel.
	%	%	%	%	%	%	%
A	0.37	0.260	0.31	.....	.....	.....	3.05
B	0.34	0.260	0.29	.....	.....	.....	5.60
E	0.10	0.012	0.33	0.008	0.034	0.056	6.01
F	0.34	0.224	0.24	0.013	0.010	0.064	6.17
G	0.566	0.338	0.49	0.016	0.019	0.064	25.74
H	0.52	0.29	0.35	.....	.....	.....	.....

The writer was informed in 1900 by Director Herr Uhlenhaut that steel plates containing less than 25% of nickel could not be rolled sufficiently smooth. The author's exceptionally able and complete paper shows that this difficulty has been overcome, and that the cost of manufacture has been reduced so much that nickel steel containing 3.5 and 4.5% of nickel may now be considered seriously as a material possessing decided advantages over carbon steel for the construction of bridges. The paper demonstrates that, just as structural steel has displaced wrought iron, it may be reasonably expected that nickel steel will ultimately displace carbon steel in structures.

Unfortunately, the steels described in Table 66 differ considerably from those dealt with by the author, and an exact comparison is not possible, but they were tested carefully by the writer with a machine shown by calibration to be accurate within 0.04%, and the deformation used for the determination of the elastic constants was obtained with Marten's mirror extensometer. The results of these tests are generally in accord with those quoted by the author, and they are summarized in Tables 67 to 70, as they may be considered to be of some interest in this discussion.

* Challis Professor of Engineering, University of Sydney.

† "Some Physical Properties of Nickel Steel," *Proceedings*, Royal Society of New South Wales, 1898; also, The Australasian Association for the Advancement of Science, Hobart Meeting, 1902.

Mr. Warren.

TABLE 67.—TENSION TESTS.

Reference letter.	Tensile strength, in tons per square inch.	Limit of proportionality, in tons per square inch.	Ratio of limit to break. Percentage.	Contraction of area at fracture. Percentage.	ELONGATION MEASURED AFTER FRACTURE.		Coefficient of elasticity, in tons per square inch.	Quality factor.
					On 6 in.	On 3 in.		
E..........	32.88	19.2	59	72	1.75	1.08	12 375	9.6
F..........	43.50	33.7	78	61	On 8″ 1.45	On 4″ 1.00	12 500	7.8
F..........	48.00	36.4	76	68	1.40	1.00	12 500	8.4
F..........	51.40	23.8	47	43	On 6″ 1.30	On 3″ 0.80	12 500	11.1
A..........	45.3	30.2	67	58	On 8″ 1.40	On 4″ 1.00	12 767	7.2
A..........	45.3	30.2	67	56	1.45	0.98	12 767	8.2
G..........	47.88	13.1	27	70	On 6″ 2.35	On 3″ 1.36	11 760	18.7

TABLE 68.—COMPRESSION TESTS OF CYLINDERS ONE INCH IN DIAMETER.

Reference letter.	Length, in inches.	Limit of proportionality, in tons per square inch.	Coefficient of elasticity, in tons per square inch.	Remarks in regard to limit E.
H..............	2	12.05	10 625	Not well defined.
H..............	2	13.30	10 625	do.
H..............	4	15 60	10 625	do.
A..............	2	25.60	11 785	do.
A..............	2	25.60	11 785	do.
A..............	4	22.18	11 785	do.
B..............	2	28.43	11 071	Well defined.
B..............	2	28.43	11 071	do.
B..............	4	20.00	11 071	do.
G..............	10	11.99	11 360	do.
E..............	10	14.50	12 705	do.
F..............	10	13.10	12 570	do.

The coefficient of rigidity was obtained by measuring the twist of a bar 15 in. long and 0.723 in. in diameter:

$$A = 11\,600\,000 \text{ lb. per sq. in.}$$
$$B = 11\,200\,000 \text{ " " "}$$
$$H = 11\,800\,000 \text{ " " "}$$

Table 71 shows the properties of 6.1% nickel steel in which the percentage of carbon has been varied, producing three qualities, mild, medium, and hard; the complete analysis was not determined.

TABLE 69.—TORSION TESTS. Mr. Warren.

Length on which strain was measured = 3 in. for A, B, and H, and 1.125 in. for G, E, and F.

Reference letter.	Diameter, in inches. d.	Number of turns at fracture.	Twisting moment at yield point, in inch-pounds.	Twisting moment at fracture, in inch-pounds.	Stress at yield point, in pounds per square inch.	Stress at fracture, in pounds per square inch.
A	0.75	$2\frac{1}{4}$	7 168	7 710	86 600	93 300
B	0.75	$2\frac{1}{8}$	7 168	9 420	86 600	113 700
H	0.75	$1\frac{1}{4}$	5 380	7 168	65 100	86 600
G	0.715	$2\frac{1}{2}$	......	10 931	......	152 578
E	0.715	$1\frac{1}{8}$	......	6 809	......	95 448
F	0.715	$\frac{1}{2}$	......	8 960	......	125 591

TABLE 70.—SHEARING TESTS.

Length of piece, 5 in.; grooved at two sections for double shear, 0.16 in.

Reference letter.	Area exposed to shear, in square inches.	Load at rupture, in tons.	SHEARING STRESS REDUCED TO SINGLE SHEAR.	
			Tons, per square inch.	Pounds, per square inch.
A	0.88	27.4	31.14	69 750
B	0.88	38.25	43.46	97 500
H	0.88	25.00	28.45	63 700
G	0.95	44.40	46.50	104 050
E	0.96	31.92	33.20	74 312
F	0.96	42.50	44.10	98 780

The results are interesting, more especially as some pieces were first subjected to repetitive tests in a rotating machine under an extreme fiber stress of 54 085 lb. per sq. in. before subjecting them to the tensile tests.

In regard to the relative resilience of nickel and carbon steels, the results obtained by the author are less favorable for nickel than for carbon steels, whereas Professor Hatt's tests are decidedly in favor of nickel steel.

The writer obtained the results recorded in Table 72 with a Marten's impact testing machine, similar to the one described in Johnson's "Materials of Construction." The specimens were subjected to a shock producing tensile stress; they were of standard form, $L = 11.3 \sqrt{a}$, the volume of the parallel portion being 0.212 cu. in. It required a number of blows to break the piece, and the extensions per blow were measured with a cathetometer.

Mr. Warren.

TABLE 71.

Description.	ORIGINAL DIMENSIONS.		STRESS, IN POUNDS.		Stress, in tons per square inch.	Limit of elasticity, in tons per square inch.	Ratio of limit to break.	CONTRACTED DIMENSIONS.		Contraction of area, percentage.	ELONGATIONS, MEASURED AFTER FRACTURE.		Local elongations.	General elongation, percentage.	Coefficient of quality.	Revolutions before testing.
	Diameter, in inches.	Area, in square inches.	Total.	Per square inch.				Diameter, in inches.	Area, in square inches.		On 6 in.	On 3 in.				
Nickel steel containing approximately 6% of nickel; mild	0.625	0.3068	22 800	74 320	33.18	23.4	71.9	0.334	0.0876	71.1	15.6	10.5	0.54	17.0	5.6	....
	"	"	22 900	74 640	33.32	22.5	67.7	0.340	0.09079	70.4	16.75	10.8	0.485	19.9	6.7	....
	"	"	22 750	74 150	33.10	22.5	68.1	0.337	0.08919	70.9	15.0	10.2	0.54	16.0	5.3	....
	"	"	22 820	74 370	33.20	22.9	69.2	0.337	0.08919	70.8	15.48	10.5	0.52	17.6	5.9	....
	"	"	22 550	73 500	32.81	20.3	62.1	0.345	0.09482	69.1	15.8	10.0	0.42	19.3	6.3	20 200
	"	"	22 500	73 340	32.74	20.3	62.1	0.345	0.09412	69.1	16.0	..	0.08	25.3	8.3	14 350
	"	"	This specimen broke in the alternating machine.							..	..	8.4	......	....	....	27 650
Ditto; medium	0.625	0.3068	31 250	101 860	45.47	30.6	67.2	0.427	0.1432	53.3	11.6	8.0	0.44	12.0	5.5	....
	"	"	31 000	101 050	45.11	30.6	67.8	0.400	0.1257	59.0	12.6	8.4	0.42	14.0	6.3	....
	"	"	30 750	100 240	44.75	30.6	68.6	0.385	0.1164	62.0	13.0	8.7	0.44	14.3	6.3	....
	"	"	31 000	101 050	45.11	30.6	67.8	0.404	0.1280	58.1	12.4	8.4	0.43	13.4	6.0	....
	"	"	31 250	101 860	45.47	32.0	70.4	0.396	0.1232	60.0	13.6	8.8	0.40	16.0	7.3	20 800
	"	"	31 250	101 860	45.47	31.7	69.9	0.407	0.1301	57.6	12.0	8.2	0.44	12.6	5.7	13 800
	"	"	31 250	101 860	45.47	28.1	61.7	0.393	0.1213	60.4	12.7	8.6	0.45	13.6	6.2	29 050
Ditto; hard	0.625	0.3068	37 400	121 900	54.42	38.6	70.9	0.480	0.18095	41.0	9.6	6.6	0.36	10.0	5.4	....
	0.624	0.30582	36 750	120 170	53.64	38.6	72.4	0.480	0.18095	40.8	9.3	6.4	0.35	9.6	5.1	....
	0.625	0.3068	37 580	122 500	54.68	39.2	74.5	0.479	0.18019	41.2	9.4	6.4	0.34	10.0	5.5	....
	"	"	37 243	121 520	54.25	38.7	72.6	0.480	0.18070	41.0	9.4	6.5	0.35	9.9	5.3	....
	"	"	37 750	123 050	54.93	38.7	70.4	0.471	0.17417	43.2	9.2	6.1	0.30	10.3	5.7	15 050
	"	"	36 275	118 240	52.78	37.8	71.6	0.433	0.14705	52.0	9.1	6.7	0.43	8.0	4.2	28 450
	"	"	36 275	118 240	52.78	37.0	70.3	0.472	0.17490	42.9	9.0	6.2	0.34	9.6	5.1	15 500
											On 5 in.	On 2¼ in.				
Ditto.	0.745	0.4359	56 200	129 000	57.6	40.9	71.0	0.620	0.3019	30.7	0.80	0.44	0.08	14.4	8.2	*
	0.745	0.4359	54 200	124 500	65.6	44.2	79.0	0.570	0.2551	41.4	0.85	0.57	0.29	11.2	6.2	†

* Tested in an oil bath at a temperature of 505° Fahr. † Tested in the ordinary way.

NOTE.—The extreme fiber stress in all the rotating tests was 54 085 lb. per sq. in.

M = bending moment.

f = intensity of fiber stress,

r = radius of specimen = 0.3125 in.

$$f = \frac{My}{I} = \frac{M}{0.775403} = 54\,085 \text{ lb.}$$

Mr. Warren.

TABLE 72.—IMPACT TENSION TESTS.
Weight of hammer, 79 lb.

Description, or reference letter.	Number of blows.	Total extension, up to the last blow before fracture, in inches.	Total extension, measured after fracture, in inches.	Mean extension per blow, in inches.	Total extension, percentage on 3 in. after fracture.	Work done in causing rupture, in foot-pounds.	Specific impact, in foot-pounds per cubic inch.
H	6	0.756	....	0.151	....	787	3 712
H	7	0.681	0.90	0.113	30.0	918	4 330
H	7	0.796	0.90	0.133	30.0	918	4 330
A	8	0.645	0.80	0.092	26.6	1 040	4 938
A	6	0.640	0.70	0.128	23.3	787	3 712
A	6	0.680	0.90	0.126	30.0	787	3 712
B	6	0.592	0.70	0.118	23.3	787	3 712
B	7	0.664	0.75	0.110	25.0	918	4 330
B	7	0.916	1.00	0.153	33.3	918	4 330
Axle steel	7	0.872	1.00	0.145	33.3	911	4 330
Tire steel	8	0.656	0.70	0.094	23.0	1 049	4 938
	8	0.692	"	0.099	"	1 049	4 938

The following experiments were made, with a hammer weighing 122.5 lb., on the nickel steel recorded in Table 72 with one blow, and the mean specific impacts in foot-pounds per cubic inch for the mild, medium, and hard varieties were 1890, 1690, and 1080, respectively.

The same machine was used to obtain the results recorded in Table 73, in which the specimens consisted of notched bars, and the weight of the hammer was 40 lb. The notch consisted of a hole formed with a twist drill 4 mm. in diameter in the specimens 20 by 20 mm. in cross-section, and 8 mm. in diameter in the specimens 30 by 30 mm.; the cross-sectional areas at the notches are 200 and 450 sq. mm., respectively. The specimens were supported on a span of 120 mm., and were subjected to a number of blows varying from 0.15 to 1.00 m.

In impact tests it is necessary to keep all the conditions constant, which is accomplished by preparing all the specimens of the same form and dimensions, and testing them in the same machine under precisely similar conditions, such as weight of hammer, height of drop, and equal intervals of time between each drop; the specific impacts will then represent the relative resistances to shock of the materials tested.

The Charpy and Guillery impact machines are now much used in the writer's laboratory on specimens having a length of 60 mm., a cross-section of 10 by 10 mm., and a square notch 2 by 2 mm. The energy absorbed by the test piece in a single blow is determined accurately, and also the angle of rupture.

The results obtained with the steels denoted by *A*, *F*, and *H*, in Table 66, are as follows:

H gave 5.2 kg. and 5.5° angle of rupture.

A " 16.0 " " 26.5° " " "

F " 21.2 " " 29.1° " " "

Mr. Warren.

TABLE 73.—IMPACT TRANSVERSE TESTS ON NOTCHED BARS.

Description or reference letter.	Section at center.		Height of drop, in meters.	Number of blows required to produce fracture.	Total deflection up to last blow before fracture.	Average deflection per blow, in inches.	Work done in fracturing, in meter-pounds.	Work done in fracturing, in foot-pounds.	Remarks.
	Breadth, in millimeters.	Depth, in millimeters.							
A..............	20	10	0.3	5	0.50	0.125	60	196.8	
A..............	"	"	"	5	0.50	0.125	60	196.8	
F..............	"	"	"	7	0.57	0.092	80	262.5	
H..............	"	"	"	3	0.15	0.150	24	78.7	
Axle..............	"	"	"	4	0.42	0.135	48	157.4	
Tire..............	"	"	"	2	0.12	0.125	24	78.7	
A..............	"	"	0.15	9	0.57	0.054	54	177.0	
F..............	"	"	"	15	0.73	0.039	90	295.3	
H..............	"	"	"	3	0.15	0.078	18	59.0	
Axle..............	"	"	"	8	0.47	0.06[illegible]	48	157.4	
Tire..............	"	"	"	4	0.15	0.052	24	78.7	
A..............	30	15	1.00	5	0.35	0.087	200	656.2	
A..............	"	"	0.50	10	0.35	0.039	200	656.2	Halves still connected.
F..............	"	"	1.00	7	0.56	0.085	280	919.3	" " "
F..............	"	"	0.50	18	0.86	0.040	360	1 180.0	
H..............	"	"	1.00	3	0.25	0.125	120	394.0	
A..............	"	"	0.50	7	0.30	0.050	140	459.5	
Axle..............	"	"	1.00	6	0.62	0.124	240	787.0	
Axle..............	"	"	0.50	12	0.58	0.053	240	787.0	
Tire..............	"	"	1.00	3	0.20	0.100	120	394.0	
Tire..............	"	"	0.50	8	0.20	0.050	120	394.0	

Unfortunately, these results on shock resistance are not strictly comparable, but, taken as a whole, they show that nickel steel possesses considerable shock resistance, at least equal to that of carbon steel. Mr. Warren.

In regard to the so-called brittle zone between 5 and 12% of nickel, this does not appear to be due to the nickel, as Rudeloff showed in 1896 that the ultimate strength, elastic limit, and yield point increase with the percentage of nickel up to 10%, after which they decrease. There was no suggestion of brittleness in the writer's experiments of 6% nickel steel, so that the brittleness found by other experimenters must have been due to other causes, such as the greater proportion of carbon and manganese used.

In regard to resistance to corrosion, the writer used a 1% solution of sulphuric acid, and boiled the specimens for four days. The results obtained showed that nickel steels containing 3 and 6% of nickel were at least as good as carbon steel. The 25% nickel steel was perfectly bright after the test.

In reference to the composition of the rolled steel given by the author in Table 4, the writer believes that a suitable material for bridge work might reasonably be expected from this specification, and the tensile strength and elastic limit should also be easily obtained.

In regard to the intensities of working stresses in bridges, the writer considers that the practice of using a suitable impact formula to express the effect of the live load, and the reduction to a total equivalent dead load is thoroughly sound, and most convenient in designing. The intensities given by the author are fully justified by the various tests of nickel steel.

The most satisfactory method of finding the safe working stress in nickel steel is to multiply the known safe working stress in carbon steel by the ratio of the limits of proportionality of the nickel and carbon steels.

The compression formulas also appear to be consistent, having regard to existing knowledge on the strength of long columns, although further experiments appear to be desirable.

The writer has read the paper with great interest, and is impressed with the author's comprehensive and thorough treatment of the whole subject. The diagrams giving the cost of different types of bridges suggest that nickel steel will probably soon displace carbon steel, in whole or in part, for the construction of railway, and for long-span, bridges.

William R. Webster, M. Am. Soc. C. E. (by letter).—The only matter in this paper the writer cares to discuss at this time is the specification for nickel-steel eye-bars. Mr. Webster

The author's requirement for bending tests on unannealed specimens, of 90° around a pin having a diameter equal to three times the

Mr. Webster.

TABLE 74.

Thickness of material, in inches.	Unannealed Specimens.								Full-Sized Eye-Bars.		
	From edge.			From center.			Percentage of elongation.	Bend.			
	Elastic limit.	Ultimate strength.		Elastic limit.	Ultimate strength.		In 8 in.	3 T.	Elastic limit.	Ultimate strength.	Percentage of elongation in 10 in.
	65 000	115 000	130 000	63 000	112 000	127 000	12	90°	....	....	....
1	64 000	113 500	128 500	62 000	110 500	125 500	14	90°	57 750	105 000	10
1¼	62 000	110 500	125 500	60 000	107 500	122 500	15	90°	56 375	102 500	10
1½	60 000	107 500	122 500	58 000	104 500	119 500	16	90°	55 000	100 000	10
1¾	58 000	194 500	119 500	57 000	101 500	116 500	17	90°	53 625	97 500	10
2	57 000	101 500	116 500	57 000	98 500	113 500	18	90°	52 250	95 000	10
2¼	57 000	98 500	113 500	57 000	95 500	110 500	19	90°	50 870	92 500	10
2½	57 000	95 500	110 500	57 000	95 000	110 000	20	90°	49 500	90 000	10

thickness of the bar, is not rigid enough to insure the proper quality of steel being used for eye-bars, or to insure care in rolling. Mr. Webster.

He starts with a leeway in ultimate strength of from 115 000 to 130 000 lb., and treats the whole matter as though one heat of steel should be rolled into bars of any thickness that might be required. Allowances are made to cover this, both in the requirements of the specimen and of full-sized eye-bar tests. The permissible variations are worked out in Table 74.

The differences in ultimate strength in specimen tests are increased to 33 000 lb., or more than doubled, the low limit for a 2½-in. bar being 95 500 lb., and the high limit for a 1-in. bar being 128 500 lb. This is too great; 15 000 lb. is enough leeway in any specification for nickel-steel eye-bars.

These specifications really allow the use of two grades of steel, owing to the wide permissible variations in ultimate strengths. The author also calls for much higher ultimate strengths and elastic limits than he obtained in any of the eye-bars he tested; and he gives what he thinks would be desirable. He refers at length to the results of tests of eye-bars that met the requirements of the specifications for the Blackwell's Island Bridge; had he based his conclusions on the results of tests of eye-bars that did not meet the requirements of that specification, he would not have specified any material up to 128 000 lb. in tensile strength.

Particular attention is called to this, as some may be misled by his specification, and have trouble with this higher tensile-strength steel.

Mr. Breithaupt.

William H. Breithaupt, M. Am. Soc. C. E. (by letter).—This paper, containing the results of investigations made at considerable cost, and extending over several years, is a valuable addition to engineering knowledge. It is a description, tabulation, and graphical exposition, in Dr. Waddell's well-known exhaustive manner, of properties of nickel steel, with a view to demonstrate its economy as a material for bridges, and to find the alloy best adapted as such material. With due appreciation of the facts brought out, however, it must be held that the argument for the general use of this material for bridges remains open to various objections.

The tests show that the elastic limit of the alloy of nickel steel used is about one and three-quarters times that of medium-carbon steel, that the ratio of respective ultimate strengths is slightly less in favor of nickel steel, and that the modulus of elasticity is about the same for the two materials. Some of the bending and shopwork tests were favorable to nickel steel and some inconclusive. The corrosion of nickel steel is probably less than that of carbon steel; the tests on this were conflicting, and do not give much definite evidence. The alloy of nickel steel used is less ductile than the ordinary low-carbon

Mr. Breithaupt. steel, and therefore does not stand drifting as well. Impact tests showed better for carbon steel than for the nickel steel used. Shop-work on nickel steel will require heavier tools throughout, as shown by the punching, shearing, reaming, and chipping tests. With an alloy containing about 4½% of nickel there is danger of brittleness, but this is not well defined.

The testing of nickel steel requires to be done very carefully. Results differ materially, depending on the size of the rolled sections and on the location of the test piece, whether from the edge, body, end, etc., of the rolled section, the difference thus revealed being much more marked than in carbon steel.

The superiority of nickel steel for eye-bars may be considered as well established. For the same work, nickel-steel eye-bars can be more than 40% lighter than if of carbon steel. With the greater uncertainty of testing, proper safe-guarding, however, would require the imposing of the old-time specification of the initial testing of each individual finished bar, and this should be to a load well above the elastic limit of carbon steel. Such a requirement was not uncommon at the time when steel eye-bars superseded wrought-iron bars, and when it was difficult to get uniform steel. It would not be practicable for compression members.

The compression tests of struts, Table 27, show the superiority of nickel steel over carbon steel as considerably more for long struts than for short ones, within the limit of set, *i. e.,* within the elastic limit of the material; while, at the point of failure, the results are relatively reversed. The values are as 2.44 nickel to 1 carbon for the 30-ft. struts, and as 1.83 nickel to 1 carbon for the 10-ft. struts, within the elastic limit, and respectively at 1.76 to 1 for short, and 1.47 to 1 for long, at the point of failure. With the practical equality of E for both materials, the superiority of nickel steel for use in compression will stand more investigation by tests of full-sized bridge members before it can be considered that definite values have been obtained for the proportioning of sections. It is a grave question whether our apparent over-confidence in steel for large compression members is not due to insufficient consideration of the fact of its relative lack of superiority in stiffness.

For smaller compression members, there appears to be little or no advantage in the use of nickel steel, and the same may be said as to floor-beams and stringers—where impact, among other things, comes into play—and for girders generally. Whether the economy in weight of long compression members would figure much more than 20% over carbon steel, must be held to remain in doubt. Such economy, however, is material and will, with the much greater saving in eye-bars, and with greater facility in its production, give a definite field for the use of nickel steel in long-span bridges, and particularly in bridges of exceptional span.

The author is entitled to the gratitude of the Engineering Profession for his timely investigations and for so well putting them on record. Mr. Breithaupt.

E. A. Stone, Esq. (by letter).—On reading Dr. Waddell's paper one cannot but be struck with the heavy task which he laid out for himself in the beginning of his investigations, and the thorough manner in which he afterward carried it out. Mr. Stone.

At the present time, when our long-span bridges appear to have reached their maximum length, a thorough investigation into the possibilities of "Nickel Steel for Bridges" would seem to be most opportune, and a material which, by virtue of its high elastic limit and ultimate strength, reduces the dead load of such bridges, and enables us to increase the maximum allowable span by 500 ft., is one which should commend itself to all bridge engineers.

As the paper would seem to prove very conclusively the superiority of nickel steel over carbon steel, from the standpoint of the economics of engineering, the question now to be answered is: Why not use it? The main difficulty, however, would seem to be the sensitiveness of the public generally to do anything in the nature of an experiment, together with the change of plant required in the bridge shops, the present machines for carbon steel not being sufficiently heavy; a change which would take place immediately, no doubt, on the demand for such a product being made.

Table 5, comparing probable shop costs, is interesting. As the items for drawing-room work, template-shop work, and laying-out work show an increase, the writer supposes the table refers to "mixed steel" bridges. It would seem that the fabrication of mixed-steel bridges might not possibly be regarded altogether with favor by manufacturers on account of the necessity of using two different grades of material in one bridge, which would increase the likelihood of error in both shop and office over those where the entire structure is made from one class of material only.

An element tending to keep up the cost of nickel-steel bridges is referred to on page 242, where the fact is stated that the high tensile strength of this material, by diminishing the size of the sections, would diminish the bridge shop tonnage, while the operating expenses would remain the same. This fact would undoubtedly be a factor in the new shop costs to be determined.

The next logical step would seem to be for some one to build a few nickel-steel and mixed-steel bridges, in order to see how the observed costs actually compare with those we now have for carbon steel. The exhaustive set of physical tests carried out would seem to be conclusive as to the suitability of the material, so that the only "non-proven" qualification would be that of actual cost. It is to be hoped that such bridges will shortly be built and that Dr. Waddell's cost diagrams will be fully substantiated.

Mr. Codron. C. Codron, Esq. (by letter).—The writer has read this excellent paper with much interest.

The results of the author's numerous tests, the observations which he has made, and the conclusions resulting therefrom—in fact, the whole of this remarkable work—comes at the proper time, and is an important contribution to the general study of nickel steel for bridges as compared with the steel ordinarily used in metallic structures.

This paper seems to be very complete, as well as most instructive, and deserves the serious attention of constructors. Little remains to be brought out with reference to those properties of the steel which, within the percentages of nickel and other foreign matters considered, best qualify the metal for the uses intended.

It is desired, however, to call attention to the fact that it would be of great advantage to be in possession of additional data relative to the manufacture of nickel steel, in order that constructors may gain confidence as to its uniformity, an essential condition for its use with the guaranty of unquestionable success. It would be desirable, therefore, to have data as to the exact choice of the raw materials and, in great detail, their mode of treatment: on the casting of the ingots, their form, weight, and whether or not they are compressed, what defects they show, and the manipulations they must undergo before rolling.

The best methods for transforming the ingots into plates or sections are also most interesting studies. It would be of great interest to possess detailed information as to the care to be taken in rolling nickel steel as compared with ordinary steel: as to the temperatures of rolling; how to conduct the drawing out of the metal, whether by hammer, press, or rolling mill, with the extreme temperatures during these manipulations, and the defects which are developed when the methods are not of the best. All these points in the study of nickel steel are most interesting, not only with respect to bridges, but also all other possible ways in which it may be used, the same methods of mill manipulation being required in all cases.

In France, there are no bridges of nickel steel; engineers have been satisfied, heretofore, in making very modest tests on beams with reduced dimensions, because it has been difficult to roll large-enough flanges. It would seem that the difficulty of rolling the shapes required in bridge construction would lead to the adoption of a steel either free from nickel or containing a smaller percentage than may be contained in steel for plates and other simple sections.

The author's numerous tests confirm the general facts which have been indicated by more modest experiments, particularly as to the coefficient of elasticity, ultimate strength, ductility, resilience, and resistance to corrosion. It cannot be doubted that for long-span bridges nickel steel will assert itself, not merely because it will permit

their extension, but chiefly because it deteriorates less than ordinary steel under the influence of the environment in which it is placed. This is a vital quality, which will lead to the use of nickel steel with as high a percentage of nickel as possible for bridges of small span also, and, in fact, for all structures exposed to corrosion. Since the corrosion becomes less as the percentage of nickel increases, the main problem will really be to manufacture steel containing a high percentage of nickel with as much ease as ordinary steel, and, at the same time, give it, not merely a higher ultimate strength, but also a high ductility. Mr. Codron.

Of the numerous tests which the author has considered, and has carried out with great perfection, the writer only wishes to offer some remarks on those of flexure or bending, which are in every case essential tests for a knowledge of the ductility, one of the most important properties of the metal, when combined with sufficient tenacity. It is strange that in bending tests it is generally considered sufficient, on the one hand, to specify the angle of bend, without even indicating the radius of curvature—which shows absolutely nothing—or, again, to fix the angle, and also the radius, in terms of the metal thickness—which is better, but yet insufficient for rapid comparison with tests involving different thicknesses and therefore different radii of curvature; and this in spite of the fact that it is so simple to obtain the important characteristic result of this test, namely, the elongation of the extreme fibers at the time when cracks appear.

This test is most valuable. It should always be made with great care, as the author has indicated by placing it among the conditions prescribed by the specifications.

When the bending is made around a mandrel having a diameter equal to n times the thickness of the specimen, while its two parts form an angle, a, before cracks appear on the extreme face, it is apparent that the process described will be sufficient for the purposes of the test, but it is useless to specify the angle, a, as the curvature of the mandrel itself sets the limit to the bending of the test piece. It may be added, however, that it would be very useful to determine the amount of elongation, and to note it in the records of the test. All that need be done, in order to obtain this value, is to draw on the extreme face of the specimen a few transverse lines, at equal intervals of say 10 mm. (0.4 in.), measure the distance between these lines at the end of the test, and then compute the percentage of elongation.

It is advantageous to use first a mandrel having a radius large enough to make the angle between the two legs of the test piece less than 90°, and then gradually effect the flattening of the specimen, preferably by machine. This method permits observation as to the behavior of the extreme fibers. The operation is stopped as soon as the cracks appear.

Mr. Codron. If care is taken to draw transverse reference lines every 5 or 10 mm., it is easily possible to note the elongation of the extreme fibers, and to commute its percentage, which may then be compared with the percentage given in a tension test to rupture.

There is ordinarily little variation between these two values. In this manner the bending test may be made with all possible care; one may utilize an apparatus for testing beams simply supported, such as is easy to arrange for the purposes of this test.

Where the bending of the specimen is continued until its parts are brought flat against each other, it may happen that the bending obtained does not give the extreme elongation which corresponds to the formation of cracks. In such a case, if it is desired to obtain this extreme value, the specimen may be notched on both its parallel opposite faces; the notches may be a few millimeters deep, and a few centimeters long, and made so as to localize the strain, and obtain a greater elongation of the extreme fibers. This procedure has been adopted successfully by M. Breuil, Chief of the Testing of Metal in the Conservatoire des Arts et Métiers de Paris.

Mills and testing laboratories should possess a special machine for bending test pieces, and for enveloping them around the mandrel until they fail, without paying much attention to the angle of bend.

The author's impact tests, although giving somewhat contradictory results, show the difficulty of obtaining uniformity of fabrication; altogether, they indicate important facts which must be taken into account in the consideration of nickel steel and its uses.

The author has been careful to call attention to this point, and to those results of the tests which are at variance with current ideas on the matter.

It would therefore be well to continue the impact tests on an extended scale, with specimens containing varying proportions of nickel, and with fabricated members of the same composition.

But the important point has been to establish the fact that the nickel steel considered possesses a ductility entirely sufficient for its use in bridges, and that it offers complete assurance against brittleness.

The author's remarks about rivets are very judicious; the writer agrees with him, particularly as regards their increase in diameter for ensuring greater strength in the riveted connection.

It is certain that, as a rule, constructors will not adopt nickel steel unless the metal is demanded by their clients.

Engineers will not hastily relinquish ordinary steel, the nature of which is so well understood and presents such great uniformity of production, for another metal such as nickel steel containing even a small percentage of nickel, the manufacture and fabrication of which are still subject to many uncertainties.

The future belongs to the metal which combines high resistance with sufficient ductility.

Mr. Waddell is to be congratulated on having undertaken these new tests for the purpose of giving more complete information in reference to the advantages of nickel steel in bridge construction. Mr. Codron.

It is hoped that these most valuable tests may serve to advance considerably the study of this problem.

W. W. K. SPARROW, ASSOC. M. AM. SOC. C. E. (by letter).—The bending tests to which the material was subjected were severe, and prove conclusively that, as regards ductility, nickel steel is satisfactory for bridge building. The British Standard Specification for structural steel for bridges, which is for a 60 000 to 70 000-lb. steel, is that the piece must withstand, without fracture, being bent over until the sides are parallel, and the internal radius is not greater than one and one-half times the thickness of the test piece. The Munich Conference recommended that the pieces be bent over a rounded edge of 1 in. diameter for all thicknesses. Mr. Sparrow.

The writer would certainly expect to find the resilience of nickel steel considerably greater than that of carbon steel, and is at a loss to account for the contradictory results of the author's tests and those of Professor Hatt, unless they can be attributed to the manner in which the tests were made.

The paper does not contain sufficient data to enable one to arrive at a very clear conception of the *modus operandi* of the author's tests. His apparatus was of the impact-hammer type, the weight of the hammer multiplied by the height of drop plus the deflection of the test piece being the work done under each blow, and the ultimate resilience of the piece the total work done in producing rupture. The rise of the elastic limit after the release of a stress in excess of it is well established, and, in cases of repeated impact, stiffening of the material takes place; consequently, a considerable amount of the work done by each blow is spent in heat due to the change in the molecular structure of the metal, and therefore the total work expended is not a true measure of the ultimate resilience. In the fourth series of tests the fall would appear to be too great, giving a high velocity of impact, resulting in the work, to a certain extent, being spent locally, since time is required to transmit the internal stress. It would also be interesting to know whether there were any means of measuring the rebound of the hammer, as such rebound would naturally have to be deducted from the total fall of the hammer in order to give an accurate measure of the work done. It is a pity that an autographic diagram of the stress-strain curves of both materials was not printed with the paper, from which the comparative amounts of work done in rupturing the test bars could be accurately computed, but an approximation can be made from other observations. Neglecting the small elastic work, the stress-strain curve may be considered to be a parabola from the yield point to the point of maximum load. Then if λ be the total elongation measured after rupture, in inches per inch, and S_y and S_m

Mr. Sparrow. the stresses in pounds per square inch at the yield point and point of maximum load, the total work per cubic inch up to rupture is nearly

$$U = S_y \lambda + \frac{2}{3} (S_m - S_y) \lambda = \frac{\lambda}{3} (S_y + 2 S_m).$$

This equation was first given by Dr. Kennedy.* According to Table 28, Appendix A, S_y and S_m for nickel steel are 56 000 and 104 600 lb., respectively; and for carbon steel, 22 500 and 58 100 lb.; the elongation, in inches per inch, being 0.173 and 0.308 for nickel and carbon steels, respectively. Solving the above equation for

nickel steel, $U = 15\,293$ in-lb.

and for

carbon steel, $U = 14\,188$ "

In other words, for 1 cu. in. of material, the ultimate resilience of nickel steel is 15 293 in-lb., and of carbon steel, 14 188 in-lb., nickel steel being barely 8% higher than carbon steel.

Of course the writer is aware that the result obtained by a static test may differ considerably from a rapid one, and that the resilience thus determined is not a measure of the capacity of the material to withstand a shock or sudden blow; but the work done in rupturing a bar, depending as it does on both the ductility and the strength of the material, is a valuable criterion of its suitability for structural purposes.

Solely on account of the refinement of the testing machine at Professor Hatt's disposal, the writer would prefer his tests, and in his opinion it is a decided advantage to test by a single blow—as was the case in the impact-tension tests conducted by Professor Hatt—than by several repeated ones, as the rupture of the bar is caused by a known quantity of work, and the indeterminate phenomena, due to stiffening and loss of work by heat, are avoided to a great extent.

It is to be hoped that further tests will be made at an early date, and the question of the resilience of the two materials finally settled, for the author's impact tests, while very satisfactory as proving the great abuse to which nickel may be subjected, can hardly be classed as such as a measure of the resilience, and, in the interests of the Profession, it is essential that a further series of tests be made. It is to be sincerely hoped that some one as self-sacrificing as the author will be found ready to perform them.

In his tests on rivets Dr. Waddell learned that nickel-steel rivets should be used in nickel-steel plates and carbon-steel rivets in carbon-steel plates, owing to nickel steel being harder than carbon steel. This being so, it would be interesting to know how he intends to make the connections in his proposed composite structures of nickel and carbon steel, for if a carbon-steel member is to be connected to a nickel-steel member, the connecting plate must be either of carbon steel or

* *Minutes of Proceedings*, Inst. C. E., Vol. LXXVI.

nickel steel, in which case there will be nickel-steel rivets in a carbon-steel plate or *vice versa*. Mr. Sparrow.

In venturing any further remarks, it would be to note the great advantages of nickel steel over carbon steel for long-span bridges, whereby it is possible to increase the maximum limiting span of carbon-steel bridges by nearly 12%, as shown by the tables, without increasing the cost per foot run; and also that the maximum limiting span of nickel-steel bridges is 28% greater than that of carbon-steel bridges, with an increased cost of 37% per foot run. The advantages in favor of the new material appear to be pretty conclusive, with the exception that many points have yet to be decided, upon which the author invites co-operation, such as the proper treatment of nickel steel to give the best results for eye-bars, the best composition for rivet steel, and the behavior of the metal under impact.

In conclusion, those features of the paper which impress the writer most are: (1) there is nothing left to chance; (2) there is an open mind all through, and no bias; (3) there are no hastily drawn conclusions; and last, but by no means least, the whole is done in the interests of the Profession, a characteristic far too rare nowadays.

B. J. Lambert, Assoc. M. Am. Soc. C. E. (by letter).—Assuming that the weight and cost curves as given by Dr. Waddell are correct, there is then furnished good evidence that for all except very light bridges, the nickel-steel structure is more economical than the carbon steel, and especially so when the cost of erection and freight rates are high. Mr. Lambert.

From inquiry among bridge manufacturers the writer has found that highway-bridge work comprises probably from 30 to 50% of the bridge output of the Central States. Until the possibilities of carbon steel are more fully utilized than at present, the use of nickel steel in structures of this class would hardly be desirable. It might serve to give the scalper another opportunity to boost his already extravagant prices, and to thin up sections that are already too sylph-like.

In smaller bridges there are generally members, the cross-section of which is determined, not from the computed stress, but from the allowable radius of gyration, minimum area, or minimum thickness. The use of nickel steel in these members would now cause unnecessary expense. It might easily be argued then that for these structures a combination bridge composed of nickel-steel and carbon-steel members would be desirable. Theoretically, this would do nicely, but in practice it is easy to see what troubles would be imposed upon the manufacturer. The exterior appearance of the rolled section would not identify its composition, and in the process of fabrication there might easily occur an exchange which later would prove disastrous.

This brings up the point which, it seems to the writer, will have considerable weight with the rolling mills and bridge companies,

Mr. Lambert. namely, the difficulty of handling the two steels without danger of confusion. This may seem more fanciful than real, but even Dr. Waddell says, in reference to assembling a certain strut, "Extreme precautions were taken at every step to keep the two steels separate."

To a certain extent this same confusion may have existed when the change was made from wrought iron to steel. It will be recalled that the late Colonel Roebling's original plans for the Brooklyn Bridge called for wrought-iron cables; but progress in steel-wire manufacture caused him to change to steel; and it is due to this change alone that the Brooklyn Bridge is now able to accommodate the increased loadings imposed upon it.

Nickel steel has been and is in a somewhat analogous position. Its superiority over carbon steel in suspension, cantilever, and long-span bridges is apparent, and, with such an enthusiast as Dr. Waddell behind it, it is thought that the objections to its use in the more common structures will finally be removed.

There are many points of extreme interest to a bridge engineer in Dr. Waddell's paper, and one cannot but be impressed by the energy and zeal which produces and then makes public this great amount of valuable information.

Mr. Marriott. WILLIAM MARRIOTT, ESQ.* (by letter).—The information given in this paper is certainly very interesting. A steel half as strong again as carbon steel makes one long for a chance to try it. We have some knowledge of the behavior of nickel rails, nickel steel in high-class machine work, etc.; but, so far as the writer is aware, its behavior in actual bridge work is not known. It seems that we require to know something of its action under "fatigue," and whether it tends after years to become crystalline and fracture.

There is also in Great Britain the great question of corrosion and consequent up-keep. It is only now being generally admitted that in this respect steel is not as durable as iron, and needs greater care in maintenance.

Steel segregates badly, and the pitting is generally attributed to this segregation. If nickel steel will corrode as badly as ordinary mild steel, and as rapidly, the pitting on the much lighter scantlings will constitute a serious danger.

As far as the writer is aware, there are at present no Board of Trade regulations which would allow of this steel being used in Great Britain with economy.

Mr. Rohwer. HENRY ROHWER, M. AM. SOC. C. E. (by letter).—Dr. Waddell's paper cannot fail to be studied with great interest by all engineers, especially by those who are called upon to design bridges, and they will find it, not only of interest, but of great value, should the conclusions reached by the author prove to be true.

* M. Inst. C. E.

The author, by his diagrams and numerous tests, shows the most effective combination, as far as he has discovered; he goes further, and, by taking into consideration the price at which nickel is being produced, shows the cost of nickel steel for various styles and lengths of bridges. He has thus contributed to engineering literature one of the most important treatises on the application of nickel for technical purposes, the extent of such application depending greatly upon the quantity of nickel mined and the price at which nickel steel can be manufactured. Mr. Rohwer.

Up to the present time, very little has been made known regarding nickel steel and its behavior when subjected to strains such as exist in bridges, though the idea of its use in bridges is not altogether new.

When the construction of the bridge across the Rhine, at Cologne, Germany, was under consideration, the use of nickel steel was not only suggested, but extensive tests were made under the auspices of the machine works of Augsburg-Nuremberg and at the factory of Krupp, who was to furnish the material. Whether the results were too meager, or of insufficient weight to warrant the introduction of nickel steel, the writer does not know; suffice it to say that it was not used, and, as far as the writer knows, the results of the tests were never published.

German engineers seem to consider it safe to use nickel steel with from 2 to 8% of nickel, for bridges, for it is claimed that such material has shown a tenacity of 65 kg. per sq. mm., a ductile limit of 45 kg. per sq. mm., and a flexibility of 18 per cent. Compare nickel, chrome and molybdenum steel mixed with small percentages of carbon, in their behavior towards acids and in their capacity as electric conductors;* also nickel steel and nickel-iron-carbon compositions in their relation to compressing and shearing resistance.†

As to the qualifications and action of nickel steel, in regard to extension, when heated, M. Guillaume has made many researches, some of which have been published.‡

The nickel steel used in the crank-shaft of the North German Lloyd steamer *Kaiser Friedrich*, has a tensile strength of 62 kg. per sq. mm. with 20.5% extension. At the break it shows fibers like those of wrought iron, furnishing proof that it will not break suddenly like ordinary cast steel.

Experiments have shown that a composition with 3.25% of Ni has 30% greater tensile strength and 75% greater elasticity than carbon steel.

According to Guillaume, nickel steel with 35% of Ni will expand only one-tenth as much as platinum and one-twelfth as much as iron

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* *Journal*, Iron and Steel Institute, 1902.

† *Stahl und Eisen*, 1902, p. 182, etc.

‡ *Comptes Rendus*, 1897.

Mr. Rohwer. and steel, combining the minimum factor of density with the minimum factor of elasticity.

According to experiments conducted in Germany, Table 75 has been compiled.

TABLE 75.

Percentage of nickel.	Expansion by heat, in mill. per 1° cent.	Density per 1°, Celsius.	Elastic modulus.
0.0	10.3	7.84	22.0
24.0	17.5	8.06	19.3
31.4	....	8.01	15.3
35.7	0.88	8.10	14.7
44.4	8.5	8.12	16.4
100.0	12.5	8.85	21.6

Rivets made from nickel steel of the following composition: nickel, 3.4%, carbon, 0.25%, manganese, 0.58%, and sulphur, 0.008%, rolled when red hot, show a limit of rupture of 3 400 at a strength of flexure of 5 900, and an extension limit of 22 per cent.*

Guillaume found that nickel steel combines resistance against corrosion with a high resistance of stresses and great malleability in case of working.

Nickel steel with 36% of Ni has the lowest coefficient of expansion known.

With the opening of the very extensive fields of sulphur compounds and silicates (chief of which is nickel pyrites of iron and copper) at Sudbury, Canada, but recently discovered, much more nickel may be produced in the future than in the past. The following quantities have been produced: 200 tons in 1840, 4 427 in 1896, 6 898 in 1898, 7 526 in 1900, 8 600 in 1902, and about 10 000 in 1905.

With the increase in the use of nickel there is no doubt that its production will increase correspondingly; otherwise, the beneficial effect of its combination with iron as nickel steel in its relation to bridges, shown so clearly and comprehensively in this excellent paper, will not be realized to its fullest extent.

Mr. Wagner. SAMUEL TOBIAS WAGNER, M. AM. SOC. C. E. (by letter).—It is very unusual that the Society should have presented to it a single paper which covers so thoroughly the properties of a structural material about which comparatively so little has been written and for which so much has been claimed from time to time. Dr. Waddell is to be congratulated for presenting so complete a series of tests.

The general use of a material for structural purposes, however, must of necessity progress but slowly, and must inspire confidence as it shows, in a practical manner, adaptability for its intended use. Its practical manufacture in commercial quantities, its adaptability to

* *Journal*, Am. Soc. Naval Engrs., 1898, p. 1038; *Stahl und Eisen*, 1899, p. 1020.

withstand modern methods in shop treatment, erection, and service in the structure, if successfully shown, will do much to give confidence and extend its use in the future. Mr. Wagner.

It took a number of years for steel to supplant iron, and the history of the early uses of steel for structures is one that it would be well to have in mind in the introduction of nickel steel. There is no use traveling over the same ground twice.

The general impression which is made by a perusal of the paper is that for some time the use of nickel steel is likely to be confined to long spans, where it is possible to take advantage of its peculiar characteristics and to keep it under careful supervision. For structures of ordinary length, its use would seem to be more questionable, as the generally recognized grade of structural steel has many properties that commend it to the user. The material which we are now able to obtain, with an average tensile strength of 60 000 lb., will be all that can be desired for ordinary purposes, until we have more data relating to this or any new material than we have at present.

The tests of a practical nature given in the paper seem to show that nickel steel of the grade tested, when compared with carbon steel of corresponding strength, is better than might have been expected. Generally, however, such tests do not show as good results as with the carbon steel, although in some cases the differences are very small. They all indicate, however, in a measure, the same relative differences that existed when steel was first used, and when the tendency was to use a high-carbon steel in order to obtain a high elastic limit and tensile strength. The practical manipulations of such high-carbon steel were not satisfactory.

The idea, of using in ordinary structures nickel steel for a portion of the structure and carbon steel for the remainder, does not appeal to the writer, for a number of practical reasons. For long spans and for structures of considerable magnitude, its use in this manner is not so objectionable.

The tests which have been made for corrosion are such as to leave the result in doubt, but, at the best, tests of this character are very unsatisfactory, and the only practical method of determining any advantage which nickel steel may have in this particular over carbon steel will be by experience in its use.

At first sight it seems surprising that the coefficient of elasticity was not higher in the nickel steel than in the carbon steel, but a comparison of tests made on different grades of carbon steel seems to indicate that this might have been expected. Being as it is, the coefficient of elasticity seems to be practically constant for all grades of steel, and nickel steel does not seem to offer any advantages for use in columns and struts.

For material which is subject to heat treatment in shop manipula-

Mr. Wagner. tions, such as eye-bars, the utmost care must be exercised when steels are used which contain from 0.35 to 0.45% of carbon. Results of such tests are clearly shown in the records of the tests of the eye-bars for the Blackwell's Island Bridge, which bars contain from 3.22 to 3.76% of nickel. It is a difficult matter to make a good showing with full-sized tests on eye-bar material which has such amounts of carbon, on account of difficulty in the heat treatment. There may not be very great differences in the ultimate strength, but the variations in the elongation and the character of the fracture are likely to be troublesome to the manufacturer. Table 60 gives the list of tests that met the specifications. It would be very interesting to know what difficulty was experienced, if any, in meeting the specifications on this large work. It is this feature of the case that appeals to the writer, especially in the use of such material. If no special difficulty was encountered, the resulting product would be reasonably sure to be satisfactory as a whole; if there was a struggle to obtain the result, some not altogether satisfactory material is likely to get into the work, although it may not be shown by the tests. Uniformity in the character of the product goes a great way toward good service in the structure, and when a manufacturer is called upon to produce some special grade of material which is out of the ordinary run of his practice, the greatest care must be exercised in order to accomplish the desired result.

Some years ago, when structural steel was in its infancy, the writer was inspecting some 60 000-lb. steel at a plant where this grade of material had never been made before. The orders were small, and, between the heats which were made for him, heats of high-carbon steel were manufactured in the same furnaces. For a long time, whenever an attempt was made to change from the high-carbon to the low-carbon steel, the first heats were never quite right. After considerable experience, this difficulty was overcome. In changing from a low-carbon steel to a high-carbon nickel steel, the same difficulty would be looked for, to a certain extent, in any existing steel mill, and for a while would probably interfere with the thoroughly uniform character of the nickel-steel product. Time and experience, of course, would reduce this difficulty to a minimum.

While the tests that have been presented are unusual in their extent, many more data will be needed before the use of such steel will be specified for spans of ordinary length. The thanks of the Profession, however, are due to Dr. Waddell and his associates for making this mass of valuable information public, and it is to be hoped that the discussion will bring forth information from those who have already used nickel steel for structural purposes.

Mr. Carpenter. A. W. Carpenter, M. Am. Soc. C. E. (by letter).—The writer wishes to express his admiration for the scope and thoroughness of

Dr. Waddell's investigation, and to acknowledge his indebtedness **Mr. Carpenter.** for access to such a treasure of test data as has been disclosed. To gather the information and data required for this paper must have been an enormous task and expense. The result cannot fail to add to Dr. Waddell's already great fame as an engineer and investigator.

There would seem to be little doubt left as to the adaptability of nickel steel for bridges, and its economy over carbon steel for long-span bridges—say, spans of more than 500 ft. For railroad bridges of the ordinary range of span, the writer has not been convinced that it is desirable to change from carbon steel to nickel steel. In the first place, he would suggest that the pound prices, given as averages for different classes of bridges on page 153, are a little high for the first three classes, at least when the location is in the States of New York, Pennsylvania, or Ohio, and would suggest the following revision, bearing in mind that the erection cost should not include in any way the cost of supporting traffic nor of dismantling old structures to be replaced, which items form a large portion of the cost of erecting railroad bridges:

Deck, plate-girder spans.................... 3.5 cents.
Through, plate-girder spans.................. 3.75 "
Riveted and pin-connected Pratt-truss spans.. 4.00 "

Accepting Dr. Waddell's assumptions otherwise, his Table 6 would be modified as shown in Table 76.

TABLE 76.—PERCENTAGES OF EXCESS COST OF CARBON-STEEL BRIDGES OVER MIXED-STEEL BRIDGES.

Type of Structure.	Span Limits, in Feet.	Least Excess.	Greatest Excess.	Approximate Average.
Single-track, deck, plate-girder spans	20 to 120	− 8.5	+ 7	− 1.5
Single-track, through, plate-girder spans	30 to 110	+ 1.5	+ 8	+ 5.
Single-track, through, riveted, Pratt-truss spans	120 to 200	+ 1.4	+ 4.1	+ 3.
Single-track, through, pin-connected, Pratt-truss spans	190 to 310	0.0	+ 5.8	+ 3.
Double-track, through, riveted Pratt-truss spans	110 to 180	+ 0.9	+ 5.0	+ 3.
Double-track, through, pin-connected, Pratt-truss spans	180 to 310	− 0.6	+ 6.3	+ 3.5

Table 76 shows that deck, plate-girder spans of "mixed" steel would cost, on the average, more than similar spans of carbon steel; and that, for other types, the "mixed" steel construction would be generally a trifle cheaper. The writer does not understand that the author claims or demonstrates that a nickel-steel structure would be preferable to one of carbon steel of equal strength, at equal cost. Therefore, he sees no argument for the use of nickel-steel, deck, plate-girder spans, and feels that there are good arguments for the use of carbon steel

Mr. Carpenter. in bridges of the other types given in Table 76, even at the greatest excess cost shown. Some of the points which appear to argue in favor of the carbon steel are the following:

Experience.—The reliability and serviceability of carbon steel has been demonstrated by years of the most extensive experience. There is a possibility that, in spite of Dr. Waddell's most thorough investigation, some seriously detrimental quality in the nickel steel might develop between the mill and the finished structure ten years or more in service, which it would not be worth while to risk for the small saving in first cost.

Availability.—There seems to be no doubt that it would take longer at this period to fill an order for nickel steel than one for carbon steel. Even with nickel steel in general use, the addition of another grade of commercial steel would no doubt result in increasing the average time for filling material orders.

Danger of Mixing Steels.—The only way to effect economy in the use of nickel steel for ordinary spans appears to be in the combined use of nickel steel for parts proportioned by unit stresses which require more material than the specified minimum sections in carbon steel would supply and of carbon steel for parts in which the minimum sections would furnish sufficient strength, including ordinary laterals, stiffeners, fillers, base-plates, etc. On page 248 the author states: "Extreme precautions were taken at every step to keep the two steels separate." As the steels cannot be distinguished by surface appearances, it would seem that the use of "mixed" steel would be attended by much trouble in distinguishing the two kinds, and in danger of getting them interchanged. A mistake in a main member might be disastrous.

Elastic Distortion.—On page 150 the author refers to the greater deflections of nickel-steel spans over those of carbon steel, but thinks this not serious. The matter of stiffness, however, has been given sufficient serious attention to warrant fixing the limits of depth to length ratios of trusses and girders in prominent specifications. Such specifications require that, in case the limits are exceeded, the sections shall be increased so that the deflections shall not exceed that of spans of normal sections and limiting depth; and these specifications apply to structures designed for carbon-steel unit stresses. With nickel steel having the same coefficient of elasticity as carbon steel, but stressed 60% higher, the deflection under live load would be about in proportion to the unit stresses. If there is any good reason in the usual specification for limiting depth ratios for carbon-steel structures, then there is as good reason to make the depth limit for nickel-steel structures such as would give equal deflection, or increase the sections to give the same result. If the limiting depth for a carbon-steel girder is one-twelfth of the span, then that for a nickel-steel girder of equal

strength would be a much larger ratio, say between one-ninth and one-tenth of the span. This factor would especially affect long girder spans, and probably was not considered by Dr. Waddell in preparing his tables of weights and costs. There are a number of other ways in which the comparatively great elastic movement of nickel steel, strained with the author's working stresses, might cause trouble, and these he does not appear to mention. One of these is the extension of floor systems in which the stringers are riveted into the floor-beam webs. Assuming a 25-ft. panel and nickel-steel chords, with a live-load stress of 18 000 lb. per sq. in., and carbon-steel chords with 10 000 lb. per sq. in. (about the author's working ratios for probable actual stresses), Mr. Carpenter.

The carbon-steel chords would extend.......... 0.10 in.
And the nickel-steel chords.................... 0.18 "

Would the riveted stringer connections stand the strain with the nickel steel? What would be the lateral bending stresses on floor beams? Again, in riveted-truss spans of equal depth, would not the nearly doubled deflection of the nickel-steel span greatly increase the secondary stresses in the riveted joints, which, if properly compensated for, would wipe out all the economy of nickel steel?

Again, what would be the effect of the excessive stretching and compressing on paint and preservative coatings? The elastic distortion due to the dead load could no doubt be provided for much better than that due to the live load, as the former would be a fixed quantity for any span after being swung, while the latter would vary from zero to a maximum and back, with the passage of every live load. With very long spans, the ratio of dead load to live load becomes large, and reaches a point at which the distortion of main members due to live load would be no greater than in carbon-steel bridges of short span. At and beyond such point the factor of elastic distortion would be mainly a dead-load matter, and could no doubt be easily provided for.

A point in connection with the properties of nickel steel which the author has not brought out quite to the writer's entire satisfaction, is the effect upon it of cold-straightening. The writer has seen some pretty bad kinks, especially in long, thin, girder cover-plates, which have been taken out in cold-rolling, and wonders if nickel steel would be as little affected by such treatment as carbon steel. The impact tests, described on page 107, which did not show to the advantage of the nickel steel, were of this nature.

The proposition to use larger rivets for nickel steel would be rather objectionable for field riveting. One of the great advantages of the nickel steel would seem to lie in the shorter rivet grips required on account of the thinner metal that would be used than with carbon steel. This alone should make rivets in nickel steel more efficient.

Mr. Carpenter. It might be pertinent here to mention that there are other alloy steels which appear to have even more remarkable properties of strength and ductility than nickel steel, and probably to cost no more. Some of these have been developed in the automobile industry. They would seem worth investigating prior to the construction of another bridge of the East River or Quebec Bridge proportions. The author has shown how such an investigation should be made, and a similar paper on vanadium steel for long-span bridges would be valuable.

The test data furnished by the author in the appendices seem to the writer wonderfully complete and valuable. Some of the results are surprising, especially the low values for elastic limit and yield point shown in many cases. It would seem that a limit should be placed on the speed of testing machines, in ordinary practice, for the determination of the yield point. The writer would like to know whether the author considers it practicable to insist, in ordinary practice, that the determination be made with the beam balancing, and if this will insure the proper low speed of test.

Regarding the tests of struts, the writer is rather disappointed that so little information is given as to the accuracy of the readings from the Phœnix machine. The calibration of this machine in connection with the cast-iron column tests conducted by the New York City Department of Buildings in 1897 showed that a correction of some 15% to the readings of the gauge on the machine was necessary to reduce the same to the actual pressures on the columns.* The author states that the figures given (for loads presumably) "are based on data furnished by the Phœnix Iron Company as to the relation existing between the mercury column and the pressure on the piston of the machine." This does not appear to take recognition of any frictional losses in the piston. It is hoped that, in view of the present widespread interest in the strength of columns, the author will take up this point more fully and remove the doubt that now exists regarding the reliability of the results he records. The suspicion that the loads recorded for the corresponding compressions are too high is somewhat confirmed by calculating the loads from the compressions, using the factor determined by the New York City Building Department tests. These tests showed that a compression of 0.001 in. in 200 in. in a Phœnix column (soft steel) 14⅝ in. inside diameter, 21 ft. long, 75.3 sq. in. section, was produced by a load of 149 lb. per sq. in., as measured by the Watertown Arsenal testing machine, the accuracy of which is unquestioned. Applying this factor to the compressions of the 10-ft. struts on a length of 90 in., the results are as shown in Table 77.

* *Engineering News*, Jan. 13th, 1898.

Mr. Carpenter.

TABLE 77.—Compressions Given by Table 51, for 10-Foot Struts in a Length of 90 Inches, and Corresponding Loads Given by the Table; also, Calculated Loads from the Calibration of the Phœnix Machine from the Watertown Arsenal Machine (viz., a Compression of 0.001 in. Requires a Load = 331 lb. per sq. in.).

Nickel Steel.						Carbon Steel.								
Strut No. 1.			Strut No. 3.			Strut No. 1.			Strut No. 2.			Strut No. 3.		
Compression, in inches.	Load by Table 51, in pounds per square inch.	Calculated load, in pounds per square inch.	Compression, in inches.	Load by Table 51, in pounds per square inch.	Calculated load, in pounds per square inch.	Compression, in inches.	Load by Table 51, in pounds per square inch.	Calculated load, in pounds per square inch.	Compression, in inches.	Load by Table 51, in pounds per square inch.	Calculated load, in pounds per square inch.	Compression, in inches.	Load by Table 51, in pounds per square inch.	Calculated load, in pounds per square inch.
0.023	11 000	7 600	0.030	12 000	9 900	0.025	14 800	8 300	0.020	11 000	6 600	0.008	11 000	2 700
0.045	18 500	14 900	0.048	18 500	15 900	0.050	22 200	16 600	0.045	20 400	14 900	0.045	22 200	14 900
0.073	27 800	24 100				0.070	29 600	23 100	0.060	24 100	19 900	0.05	24 100	16 500
0.090	34 200	29 700				0.083	35 100	27 400	0.068	25 900	22 500	0.06	25 900	19 900
0.110	40 700	36 400	0.113	38 900	37 400				0.080	29 600	26 500	0.062	26 800	20 500
0.120	44 400	39 600	0.120	41 600	39 600				0.092	33 900	30 400	0.066	29 200	21 800
0.130	47 200	43 000	0.126	43 500	41 600							0.075	31 800	24 800
0.142	50 900	47 000	0.143	46 300	47 200							0.08	33 700	26 500
0.150	54 600	49 600	0.151	49 900	50 000									
0.160	58 500	52 900												
0.170	61 900	56 200	0.162	55 500	53 500									

Compressions below horizontal lines were obtained by subtracting "permanent set" values from "temporary shortening" values in Table 51.

The results of tests on 30-ft. columns show comparative calculated loads larger than recorded loads. It is thought that the 30-ft. columns were too slender to give reliable results by the calculation method.

Mr. Moisseiff.

LEON S. MOISSEIFF, M. AM. SOC. C. E. (by letter).—To one familiar with the scantiness of information on structural nickel steel available in any language, the great value of Mr. Waddell's paper is apparent. Several papers, based on a few laboratory experiments, comprised all the published information until a few years ago. This was supplemented by some semi-confidential replies to inquiries from mills and producers of nickel, which were based more on opinions and hopes than on demonstrated facts.

When, in 1902, or about a year before Mr. Waddell began his experiments, Gustav Lindenthal, M. Am. Soc. C. E., then Commissioner of Bridges of New York City, wished to utilize the greater strength of nickel steel in the design of chains and stiffening trusses for the proposed Manhattan Bridge, and partly for the eye-bars of the Queensboro (then Blackwell's Island) Bridge, this was the state of affairs: At the request of the Department of Bridges, some melts of nickel steel were then made, and a few eye-bars were tested. The information thus obtained served as a basis for the Queensboro Bridge specifications, which covered eye-bars and pins only. Judging from the results of the eye-bar tests, as contained in the records of the Department of Bridges, these specifications have, with some minor changes, produced very good material.

The lack of information on the behavior of nickel steel in the various processes of fabrication, its effect on the cost of such fabrication and on the efficiency of the tests in use in bridge shops, was felt by engineers who wished to avail themselves of the new material, and by the producer who desired to bring it into the market. It was to supply the necessary information to the engineering world that, the writer believes, Mr. Waddell began his series of experiments.

The researches, as planned by the author, were well laid out to cover most of the information needed for the designing of bridges and the writing of modern bridge specifications. It is to be regretted that the intended series of "small special melts of nickel steel containing varying proportions of nickel, carbon, and possibly manganese" was not carried out. Such a series of melts would have settled, for some time at least, the question of the best composition of nickel steel, and would have much strengthened the author's conclusions. It will be comparatively easy for the designing engineer to get now and then some special tests on points of fabrication, or on the efficiency of some details of nickel-steel members, but he will encounter difficulties when asking for special melts, and will sometimes have to follow advice which is not always disinterested.

It is not, of course, to be expected that any single paper, in a virgin field such as that of nickel-steel tests, can cover all points and satisfy all inquiries. In due time special tests will, presumably, be made of one or the other qualities and applications of the new

material as it comes into use, but only the real test of experience will decide matters fully. Meantime, this paper furnishes enough information to enable the engineer in need of a high-resistance steel for structural purposes, to utilize the advantages of nickel steel within safe limits. **Mr. Moisseiff.**

Whether or not one will agree with the exact composition of the nickel steel proposed by the author, or with the working stresses recommended by him, it must be admitted that he has created a working basis, both for the design of nickel-steel bridges and for further experiments. His comparative cost diagrams demonstrate at sight the economy of nickel-steel bridges within the extra charges for this steel computed by him. It is quite possible that a heavy demand for this material may raise the price of nickel above the estimated extra charge of 2 cents per lb. of nickel steel, but the increase of cost would check itself automatically by the decrease in demand. A glance at the cost diagrams for the longer spans, however, shows that, as may be expected, even a considerably higher extra charge would still be found to be economical.

It has been the writer's fortune to make use of nickel steel in three cases, each of which differed essentially in the functions to be performed by the members, and also in the reasons for adopting nickel steel as the proper material for them. To the writer's knowledge, these are the only instances, up to the present time, of the use of nickel steel in main members of bridges. It also happens that these three instances are fairly illustrative of the conditions which may lead to the use of nickel steel, even aside from economical considerations.

The Queensboro Bridge was the first in which nickel-steel members were used. The great weight and capacity of this bridge required in the chords sectional areas of unprecedented dimensions. At that time it was not thought desirable to demand from manufacturers eye-bars wider than 16 in., and the excessive width of the tension chords, together with the long pins, was the primary cause which led to the adoption of nickel-steel eye-bars and pins. The resulting economy, while making the attempt desirable, was not deciding in this case. The writer's connection with the design of this bridge extends only as far as the original contract design.

The Manhattan Bridge affords a more important example of the advantages of a high-resistance steel than the preceding. This bridge will have the greatest capacity of any long-span bridge yet built. It is designed for four rapid-transit tracks, four surface-car tracks, a 35-ft. roadway, and two footwalks, each about 11 ft. in width. This means practically fourteen lines of traffic. For this traffic a "working" load of 8 000 lb. and a "congested" load of 16 000 lb. per lin. ft. of bridge have been allowed in the computations. Under "congested" load

Mr. Moisseiff. is meant a moving load of maximum density, or as dense as the dimensions of cars and vehicles will permit. The "working load" has been assumed as one-half the "congested" load; or, in other words, it has been assumed that trains, cars, and vehicles will have to operate within their own distance apart. It is well known that for a practically uniformly distributed load over the entire bridge, the stresses in the stiffening trusses will be, as a rule, comparatively low, and that they will attain their greatest values for certain partial positions of loading. To get the greatest stresses in the stiffening trusses, the fourteen lines of traffic of this bridge would all have to stop at the theoretical load limits, and the temperature would have to be simultaneously at one of the extreme deviations from the "normal" assumed for it. The probability of ever realizing this condition is very remote indeed. But, in a structure of such magnitude and importance, it is good engineering to provide for any possible, if even not probable, condition, so that the structure may be then not only safe but uncrippled. Proportionately high unit stresses may then be allowed for such greatest "congested" load stresses.

To take care structurally of the large stresses deduced from the above extreme conditions, the stiffening trusses would have to be deep and unsightly, or contain very heavy chords. In either case, the moment of inertia of the trusses would be increased, which would in turn increase the stresses, both due to temperature and to moving load, without any appreciable benefit to the cables and towers. It thus became apparent that stiffening trusses of a limited moment of inertia are desirable, and that only a high-resistance steel, such as nickel steel, will satisfy the requirements. Here, again, the economy to be realized in the cables and towers was an additional inducement to adopt nickel steel, there being more than 8 000 tons of it, but the primary cause, as previously explained, was the need of a reliable high-resistance steel which would usually be stressed low but which might possibly need at some time its full strength.

The third instance of the use of nickel steel was in connection with certain reinforcing work on the Williamsburgh Bridge. The writer had found that, due to the increase in weights of cars and other reasons, the rocker posts at the ends of the stiffening trusses and their supports required strengthening. The necessary supports were provided by the erection of what is practically a fifth leg in the main towers. The rocker posts, however, had to be of the same dimensions as the existing ones in order to fit in the rather cramped space. It was a simple thing to replace them by nickel-steel rockers and pins of almost identical dimensions, and to increase thereby their strength by 50 per cent. In this case the cost of the material did not enter into the consideration at all.

The writer believes that these three instances demonstrate fully the

desirability of a reliable structural steel of high resistance in many important cases, aside from economical considerations. Mr. Moisseiff.

The writer does not agree with Mr. Waddell as to the desirability of making the floor systems of bridges of nickel steel. The design of the floor system is frequently determined by other considerations than mere theoretical strength. The stiffness of beams thus often determines their dimensions, and minimum sections are not infrequent, even in what is known as "structural steel" beams. Buckle-plates, guard-rails, etc., will remain of the same weight. It certainly will require cheap nickel steel to realize much actual economy in nickel-steel floors.

James C. Hallsted, M. Am. Soc. C. E. (by letter).—The writer is very much impressed with the large amount of labor Mr. Waddell has performed in gathering information and preparing it for this paper. He has certainly added a great deal of very valuable information on nickel steel, and deserves the thanks of the Engineering Profession. He has demonstrated the advantages of nickel steel in eye-bars, for heavy spans, and has pointed out the possibility of its economic use in heavy compression members. Mr. Hallsted.

The writer is not as sanguine as the author as to the advisability of using nickel steel in ordinary bridge work. He has clearly shown the difficulties in the way of its use, but, in the writer's judgment, he has underrated their importance.

A sample order could readily be made by the mills, in some such way as they can turn out rails and ship them to a foreign country at a less rate than they sell them for domestic use. The cost of this sample order is not necessarily a criterion of the cost that would prevail if nickel steel were to be made as a general thing in the mills.

The wear and tear on the equipment of a rolling mill in rolling the harder steel would not be evident in rolling a sample or an occasional order, as would be the case if the mills were constantly rolling nickel steel.

Further, the comparative cost of making eye-bars in nickel and in carbon steel is not representative of general structural work. Large forgings do not tax the wearing quality of the equipment like punching, shearing, reaming, and drilling; and the shopwork on eye-bars is much less than on built members. A difference of 1.5 cents per lb. for the two grades of steel does not appear to be enough, especially when 1 cent of this is for the raw nickel. Entering into the use of nickel steel there are many factors which would increase the cost to more than that of carbon steel. Some of these will be mentioned.

There are apt to be many rejections at the mills, not only because nickel steel is a new material and demands more expert treatment, but also because the requirements must be strictly adhered to if the high units are to be justified. Comparatively high rejections mean greater

Mr. Hallsted. cost; they also mean scrap. To use nickel scrap economically, there must be created a demand for nickel-steel castings, and there are in structures few places where nickel-steel castings are required.

The rolling of nickel-steel shapes has scarcely been tried. The ramifications of the various shapes make their rolling more difficult in hard steel than in soft steel. Wear or breakage on equipment, reduced output, imperfect shapes, all mean increased cost. The thin webs of standard channels would be especially troublesome in the finishing passes; and if the thin-webbed channels be avoided for heavier weights, economy is lost.

Punching is harder on the tools in nickel steel than in carbon steel. Mr. Waddell suggests the use of larger rivets, so that the holes, being of larger diameter, will be less destructive to the punching tools. This could scarcely be done in the flanges of channels, as the standard holes now used are generally the maximum for safety. In the general run of built members made of channels, the holes are nearly all in the flanges. Reamed work is a practical necessity in nickel steel. Sub-punching in channel flanges is scarcely a possibility, because of the small size of the holes. If shops are driven to drilling, with the hard usage that drills will get in nickel steel, shop cost will go up by leaps and bounds.

Larger rivet holes in tension members mean greater reduction of net section, another dash in the scale pan which will weigh against the economy of nickel steel.

In the matter of inspection and tests, costs will also run up. It would be harder to keep track of nickel-steel rollings among a lot of carbon-steel rollings of the same shapes, and greater care would have to be exercised in separating the nickel steel, because of the menace to a structure which would result in using carbon steel where nickel steel was intended.

Even in a mill where only nickel steel is manufactured, the cost of tests made according to Mr. Waddell's proposed specifications would be great. For example, the elastic limit is the load which produces a permanent set of 0.01 in. in 8 in. To make this determination the machine would have to run very slowly; it would have to be stopped and reversed many times near the elastic limit in order to see whether the specimen had a permanent set of 0.01 in.; very careful measurement would be required to determine just when this 0.01 in. had been reached. The time consumed in making the determination of the elastic limit on one nickel-steel test, by this method, would suffice to make several complete tests on carbon steel. To stop or reverse the machine in testing eye-bars would also add to the cost and trouble.

There is no doubt that a manufacturer will add to his price on a job on which there is inspection, especially if he thinks it will be

rigid, and it would be dangerous to use nickel steel without rigid inspection. Much as inspection is desirable on all work, the fact cannot be ignored that a large part of the output of the shops is not inspected. Work of this class would scarcely be attempted in nickel steel, so that there would always be stock angles or beams of carbon steel lying around, which it might be "necessary" to use in a nickel-steel structure, if the ordered material were delayed or defective. Mr. Hallsted

Mr. Waddell has shown that, with nickel steel, all the shop processes are more expensive in labor, require better tools, and are harder on tools, than if carbon steel be used. It would appear that better shop equipment and special shops will be required, if much nickel-steel work is to be attempted. All these things mean higher cost, and seem to mean it to the extent of more than 0.5 cent per lb.

As an example of the difficulties and expense that would result in a general use of nickel steel in riveted work, Mr. Waddell's statement on page 239 will be quoted:

"The speed, therefore, with which the carbon steel was drilled was 1 in. in 35 sec., and the nickel steel, 1 in. in 52 sec. A blue-chip tool, in ordinary work, would last half a day without sharpening, whereas, if used for 5 or 6 min. on nickel steel, it is necessary to sharpen it."

There can hardly be said to be a demand for a material having the qualities of nickel steel, except in rare cases. The one feature in which nickel steel excels carbon steel is in its higher elastic limit and ultimate strength, with the possible exception that it appears to stand weather somewhat better. Nickel steel is not as ductile, not as tough, not as resilient as carbon steel; it requires more careful and more skilled treatment, and cannot stand the punishment that carbon steel will.

The relative dead weight of a nickel-steel and a carbon-steel structure is so small that there would be little difference in the stresses of an ordinary span. There are fixed weights, such as the floor, the lattice bars, tie-plates, etc., which would be the same in either, so that the relative sectional areas would not represent the comparative weights. There can be little saving, therefore, in the weight of a structure of ordinary dimensions, if, for the time being, we lay aside the diminished sections due to higher unit stresses. It can scarcely be said, then, that a demand for nickel steel exists, based on the saving in weight, from the standpoint of stress in members for ordinary structures.

In very long spans the dead weight of truss members is a large factor in the stresses on those members, and anything that will reduce that dead weight is economically advantageous. In Europe, some years ago, a suspension bridge was built with chains made of eye-bars cut out of high-steel plates. Nickel-steel eye-bars would have met the

Mr. Hallsted. demand created by the design of this bridge most admirably, and would have meant a large saving in expense. Nickel-steel eye-bars would be economical, no doubt, in spans of moderate lengths, if the tonnage justified a special order; and it may come into general use for eye-bars for all spans.

In compression members, nickel steel would possibly be economical for heavy short members. In these, direct compression plays a large part, and bending stresses are almost eliminated. Slender members in hard and soft steel approach the same ultimate strength as the ratio of length to radius of gyration increases. It can be shown that the load of Euler's formula, instead of being the compression which a column may sustain at any deflection within the elastic limit, as commonly stated, is actually the absolute maximum load that the column will carry. This load will produce failure in a column, no matter how high the elastic limit. This may appear startling. For proof the writer refers to a paper on the subject by Mr. Edward Godfrey.* This being the case, slender columns in nickel steel will approach equality in compressive strength with carbon-steel columns, so that for slender members the relative advantage of nickel steel becomes still less. It is not an accident that Mr. Waddell's tests on columns in nickel steel at $\frac{l}{r} = 27$ averaged 75% greater in ultimate strength than similar columns in carbon steel, while those on nickel-steel columns at $\frac{l}{r} = 81$ averaged only 47% stronger. The writer does not understand, however, why Mr. Waddell uses $30\,000 - 120\,\frac{l}{r}$ in his compression chords, in view of the facts shown by his tests. This, as compared with the carbon-steel formula $(16\,000 - 70\,\frac{l}{r})$, gives an allowed load 90% greater at 27 radii and 96% at 81 radii, against 75% and 47%, respectively, as found by test.

The writer thinks that Mr. Waddell's base of 30 000 in the compression formula is too high. It is a factor of about 2.6 at 27 radii, and of only 2.2 at 81 radii, according to his tests. The factor of safety for tension members is more than 3, and tension members are not affected by imperfect shopwork or alignment in anything like the amounts suffered by compression members.

If a truss in nickel steel is made of more slender compression members than one in carbon steel, these slender members will approach nearer the point where carbon steel and nickel steel have equal strength, and the economy of nickel steel tends to vanish.

The impact test of materials has been developed with the idea of demonstrating two qualities of a material. These are, first, to ascer-

* To be published soon in the *Railroad Age Gazette*.

tain whether the material is in an abnormal or dangerous state, that is, whether or not it is brittle; and second, to determine its resistance to shock under working conditions. **Mr. Hallsted.**

Impact tests have been conducted in one of two ways, either by the single-blow method or by the repeated-blow method. Both these methods of testing show unquestionably whether or not the material is brittle.

The repeated-blow tests are designed to determine the resistance of the material to shock under conditions which will approximate the actual conditions of service. In the writer's opinion, they do not furnish this information. All repeated-blow tests, of which he has knowledge, have been essentially to test the toughness of the material rather than its ability to resist repeated blows. In every test, each of the repeated blows has stressed the material much beyond the elastic limit or yield point, and the second and third blows, whether upon the same side of the test bar or upon the reversed side, have simply carried the deformation further, until ultimately the material failed under the test more as a result of the lack of ductility than anything else.

Much better would be a vibratory test such as has been made on stay-bolt iron for many years. In such tests the plastic deformation of the material is comparatively slight, and, for this reason alone, the test would more nearly demonstrate the comparative value of two materials in service.

In November, 1908, a great many data regarding impact tests were presented before the Institute of Mechanical Engineers. These tests were made on the various impact testing machines which have been on the market in recent years, machines in which every endeavor has been made to obtain as great a refinement as possible. Those used were the Seaton and Jude, the Fremont, the Izod, the Kirkaldy horizontal, and the Kirkaldy vertical. A careful series of tests was made with the idea of comparing the action of these various machines. In most cases it was found impossible to get any consistent results, and, worse than that, it was found that no one of the machines gave consistent results, even on identical specimens of steel. On one machine the results varied from 4 to 60% and more.

The irregularities disclosed by the different methods of testing have been found to be due not to the lack of uniformity in the material, as was suggested by some of the investigators, but largely to defects in the method of testing, and it seems reasonable to conclude that the method of making impact tests and the results obtained should not be relied on by engineers to differentiate between the physical properties of different materials. This conclusion was reached after careful tests on the various types of machines mentioned. From these considerations, it seems to the writer that the impact test is

Mr. Hallsted. only of value where a rapid method of discovering the brittleness of material is desired.

In Mr. Waddell's paper, it is not surprising that inconsistent results were obtained on a crude apparatus, when it is considered that even on the most refined machine the same inconsistencies occur. The writer would urgently recommend that something be done in the way of a vibratory test.

It is his opinion that reaming a rivet hole slightly, or planing a sheared edge a very small amount, say ¼ in., does not remove the effect of punching or shearing, though, without question, much good is done thereby. Some tests made by the writer several years ago on steel 1 in. thick and having a tensile strength of 70 000 to 80 000 lb., went to show that shearing lessened the ductility for about 3 in. from the edge.

As to the absence of high phosphorus, the bending test is not conclusive. The writer has made beautiful bends with 0.16% phosphorus. As with the drift test, a great deal depends on the operator.

In conclusion, the writer has stated his ideas on the use of nickel steel, not with the intention of detracting from Mr. Waddell's excellent paper, but to throw another light on the subject. He has emphasized chiefly the features affecting the use of nickel steel as they appear to one whose daily concern is in gauging and passing on the excellence of the work in the shops and mills. Only time and the developments of the qualities and uses of steel in tools will determine the place of nickel steel in structural engineering. To the writer it appears that tools, as made at present, are not capable of manipulating economically a metal having the properties of nickel steel.

Mr. Arnodin. F. Arnodin, Esq. (by letter).—Whatever individual opinion may be entertained on the subject of nickel steel and its use in construction, it must be recognized that this paper, with the numerous experiments it records, constitutes an important treatise, upon which the author is to be congratulated.

Mr. Waddell discusses mainly the determination of the proportions of nickel and carbon which will produce a steel of the greatest strength, combined with the highest elastic limit, and a malleability enabling the metal to stand easily the stress of shop manipulation. In order to achieve this three-fold purpose, he has furnished a large number of analyses and comparative tests, from which the chemical and physical properties of the material may be determined with the greatest possible accuracy.

This paper, then, is a laboratory work, and the writer, not being an expert in this branch of the subject, will leave the discussion of such features to metallurgical and chemical engineers.

However, inasmuch as the paper considers the construction of great metallic structures, especially bridges, the problem does not depend

entirely on the qualities of the steel to be used, because the kind of members, their connection, and the manner of stressing the metal, whatever the latter may be, are of equal importance. Only by harmonizing these with one another, and with the qualities of the metal, can the safety, the economy, and the progress desired be realized. It is impossible to attain these ends, if each of these considerations is treated apart from the others. Mr. Arnodin.

Accordingly, this contribution to the discussion is that of an engineer and constructor who considers the use of nickel steel in bridges from the standpoint of the safety, economy, and durability of the structure.

Disadvantages Due to High Resisting Capacity.—At the outset, it should be stated that it may be dangerous to look for high resistance in the metal used for bridges, for, in order to profit by this high resistance, and build structures of proportionally lighter weight, there will be many members having small cross-sections compared with their lengths; and therefore they cannot well resist the stress due to column flexure which is so dangerous in metallic structures. Such light sections, furthermore, are at a disadvantage with respect to oxidation. which attacks the lighter member with the same intensity as the larger one, and, consequently, will ruin the former far more rapidly than the latter.

Again, a light structure of a given span and strength of material will be influenced more by the moving load—the load of heavy trucks, locomotives, trains of cars, etc.—than a heavier structure with members of greater cross-sections. This is expressed quite neatly by the following figure: "The fly, when it alights upon a spider's web, causes more dangerous deformations than when it walks upon a table."

In short-span bridges the deformations are small and have very little effect on the structure, but they acquire a dangerous importance in long-span bridges, especially when the form of the structure is such that these deformations induce secondary stresses, which are often difficult to analyze, and which the regular static computations do not take into account.

Again, the engineer who has at heart the future of his structure, must, from the very inception of the design, consider the proper relation between the fixed loads of the structure and the traffic it has to carry. It is self-evident that, other things being equal, this relation will be more favorable with the less resistant carbon steel, than with the stronger nickel steel.

There is no doubt that, with nickel steel, one may build as heavily as with carbon steel, and, accordingly, gain greater safety. From this point of view, there is no doubt that, if the cost of each material was the same in all cases, the preference would have to be given to nickel steel, the superior qualities of which have been so well demon-

Mr. Arnodin. strated by the author's experiments. But the prices are different, and, in order to compensate for this difference, a higher strain is imposed upon the nickel steel, so that the final cost may not be increased. Accordingly, when it is used, the members are lighter, and the safety and durability of the structure are diminished.

These remarks apply particularly to members in compression, where there is danger of column bending and secondary stress, but later it will be seen that they are not justified with regard to tension members. The writer is pleased to find that Mr. Waddell has arrived at the same conclusion by methods of deduction other than those which are about to be presented; and that, in order to realize economical construction, he advises mixed structures, in which the compression members are of carbon steel and the tension members of nickel steel.

The Danger Due to Column Flexure Stresses.—Previous to the investigations by Rankine and Considère, engineers did not compute the stresses due to column flexure; as a matter of fact, they did not possess sufficiently well demonstrated theoretical formulas to effect such computations. It was only by mere estimate, and by reliance on their personal experience, that they gave the members the shape and dimensions which appeared to them able to resist these stresses. But now that they have been startled by the number of great disasters which have occurred in various parts of the world, such as that of the Quebec Bridge over the St. Lawrence, the Tay Viaduct (England), the highway bridge at Morava (Russia), the temporary viaduct at Tarbes (France), etc., they are learning to be more careful, and every engineer, worthy of the name, computes with extreme exactness the stresses due to column flexure. In France, the designers of structures for the use of the public are positively compelled to do so by the Government, and a law, dated August 29th, 1891, provides that the maximum stresses for steel shall not exceed 16 500 lb. per sq. in. for main members, column flexure and wind effect included.

In fact, even 16 500 lb. per sq. in. must be considered high for large structures. The writer, in his practice, has never dared to approach it in main members which have no intermediate supports.

In truth, the assumptions involved in the deduction of the formulas are not always fully realized in practice. An infinite number of circumstances may influence them unfavorably. In order not to extend this discussion uselessly, only two instances will be cited.

It is assumed, for instance, that a member in compression is supported normally and uniformly on its base. There is nothing perfect on earth, however, and, in a member of large cross-section, it always happens that an angle or some other point of the section carries a disproportionate strain, resulting in an unequal distribution of stress, which, again, induces flexure; and it is known that when buckling has begun, the loss of resistance of the member continues rapidly until final failure.

Moreover, in built-up members, like the chord which failed in the Quebec Bridge, provision against column flexure is ordinarily made by using the radius of gyration of the entire cross-section, without considering that each piece, of the several composing the member, owing to its small depth, is subject to the danger of individual buckling, which will result disastrously, step by step, in the buckling of the entire member. Everything points to the belief that these were the two initial causes of the Quebec disaster. It seems to the writer that it is chiefly in cases of this kind that nickel steel is out of place. Mr. Arnodin.

English engineers, more than all others, discount the effect of these disadvantages, by allowing a low modulus of stress, and using very massive sections, following therein the traditions of R. Stephenson and Brunel. The English are great users of metal, and have learned, better than any other nation, how to produce it cheaply and in large quantities, and naturally are trained in the art of using it.

On the other hand, French, German, and American engineers see the advantages of elegance and lightness. To enjoy these advantages, it is necessary to design with great precision, and leave nothing to approximation and the errors resulting therefrom.

More than all others, American engineers have shown themselves to be daring in this tendency to extraordinary lightness. Many think that they have gone too far. The writer came to this conclusion when he was informed that the specifications for the monumental Blackwell's Island Bridge authorize, for nickel steel, a maximum stress of 30 000 lb. per sq. in. for the regular load, and 39 000 lb. for the congested load; and, for carbon steel, 20 000 lb. for the regular load, and 24 000 lb. for the congested load. He is of the opinion that such stresses are much too near the elastic limit of the material, in order to have the margin of safety which all public works should possess, and where the original provisions are so often exceeded by unforeseen and manifold causes, among which may be cited as the most frequent:

First.—Increase in the moving load for which the structure was designed, made necessary by the peremptory demands of traffic, such, for instance, as compel railroad companies to have recourse to heavier and still heavier rolling stock.

Second.—An erroneous valuation of the principal stresses due to (column) flexure. This was the case with the Quebec Bridge, where, as Professor Résal has shown, immediately before the collapse, the plates of the chord which failed must have reached a stress, which, including flexure, amounted to 28 500 lb. per sq. in.,* instead of the 6 000 lb. which had been estimated for this point of the truss. In fact, Professor Résal's computations would show a greater stress than 28 500 lb. per sq. in., but he admitted the assumption of uniform distribution of stress over all the plates of the member, whereas it may be taken as certain that such a uniform distribution cannot exist.

* Encyclopédie des Travaux Publics, Vol. I, p. 582, Ch. Béranger, Editor, 1908.

Mr. Arnodin. Third.—Increase of the fixed load, resulting either from errors in the original weight estimate or from strengthening the structure during the course of erection. This is the case with the Blackwell's Island Bridge, where the expert engineer investigators, Messrs. Boller and Hodge, find that in certain members the calculated maximum stresses, with the weight of the structure as completed, reach 49 000 lb. per sq. in. for nickel steel instead of 39 000 lb., as originally specified, an increase of 26%; and for carbon steel, 35 500 lb. per sq. in., instead of 24 000 lb., an increase of 47 per cent.

These examples, important in their significance, show that it will be wise to limit the maximum stresses in the metal, whatever its nature, to stresses much below those used in the United States, particularly for compression members in which the secondary stresses are difficult to analyze and calculate with accuracy.

Then, also, it is necessary to be so much more careful with compression stresses, since failure occurs as soon as the elastic limit is exceeded, whereas a tension member will continue its service long after that has taken place.

Members in Tension.—In members of this class the science of the engineer is simplified considerably by the fact that he needs to concern himself only with the main stresses, as secondary stresses and flexure are naturally eliminated by the tension itself.

It follows, from the author's tests, that the most important property of nickel steel is its tensile resistance. Therefore he advises taking advantage of this quality by using the metal principally in tension members.

This is equally true for carbon steel, and even for wrought iron. It is a fact which has already been demonstrated by a number of clear-sighted men. It is surprising, therefore, to note how little advantage engineers of all countries have taken of this chief quality of steel, that is to say, its tensile resistance; for, in nearly all metallic structures, it is the compression member which predominates.

If it is necessary to show by facts the superiority of the tension member, it suffices to give as an example the Brooklyn Suspension Bridge, 26 years old, which continues in service in spite of the fact that on many occasions its cables have had to carry stresses which exceed by far those for which they were designed.

The point is equally well proven by the numerous photographs of the Quebec Bridge—after the failure—which show very clearly that the eye-bars of the top chord are the members which suffered the least from the many strains occasioned by the fall, even though these very members were subjected to the greatest strains. A number of these eye-bars can be used again without any repairs.

Lessons of Experience.—The Quebec Bridge disaster, and also the errors of computation in the Blackwell's Island Bridge, furnish food

for serious thought, and make it necessary that means be found to prevent, or at least to decrease the chance of, their recurrence. The engineer has a duty to perform—even as that of the physician is to check the progress of a fatal malady—and this is indicated by the fact that the investigating commission, consisting of Messrs. H. Holgate, J. G. G. Kerry, and John Galbraith, found that the errors made were not caused by insufficient technical education or by negligence, malice, or an excessive desire for economy. There is reason, then, to think that there is something wanting in the science of the engineer. **Mr. Arnodin.**

This want does exist, perhaps to a greater extent in America than in Europe. The engineer devotes himself to his studies, and to extremely laborious computations, in order to valuate the stresses in the different members of the structure he builds; then, as soon as the structure is completed, he loses sight of it, and confides its maintenance and supervision to a subordinate who is incapable of observing whether the theoretical calculations are realized in practice.

He thus resembles the captain of a vessel, who, before setting out on his voyage, carefully adjusts his compass, computes his bearings, and tests his steering apparatus, but who, as soon as the anchor is raised, leaves his ship, and entrusts its safety to an ordinary sailor, who, perhaps, has been trained to keep on a course as indicated by the compass, but who would be entirely at a loss as soon as unforeseen circumstances, such as wind, currents, etc., set in to divert the ship from its course. Such a captain would be considered guilty of negligence. Is the engineer entirely free from this reproach?

The French Government was the first to take up this matter. Again and again it has called upon its engineers to make frequent inspection tours to all the public works of their districts. It even orders them to revise their theoretical computations, so as to take account of the changes which have been made in the structure itself, or in the traffic upon it, since its completion. In other words, it compels the captain to make his observations at stated intervals, in order to verify his position. In this way the number of possible disasters is limited.

Is that sufficient? The writer does not think so; at least, not for structures of such great size as the Forth Bridge, the Quebec Bridge, the Blackwell's Island Bridge, and a number of others, principally in the United States. For, if there should be any error in computation or any unnoticed movement of the foundations under the supports, or any error in the assumptions, the safety of the bridge may become impaired, unless some visible sign gives warning.

In the writer's opinion, a large, well-planned metallic structure should be provided with an "indicator" for showing the stresses, just as a well set up steam generator has a gauge to indicate the pressure,

Mr. Arnodin. or as an electric conduit must be provided at certain points with volt meters to give the intensity of the current, and ampere meters to show its quantity, even if the boiler or the electric conduit possesses all the desired capacity for the service imposed on them.

These opinions may possibly clash with adopted ideas, or routine, but that is the fate of all new things. However, this ought not to stay the engineer, who, by his very profession, is a man of progress.

It remains to demonstrate that these ideas may be realized.

The writer shows here, for the first time, that, in order to study his structure, the engineer has only to introduce a "flexible member" [*pièce souple*] at some proper point of the bridge. This flexible member will act as an indicator to show, by indisputable external signs, what actually occurs in the structure at a given point, and, consequently, will enable the engineer to deduce, either by simple composition of forces, or by a graphic diagram, what takes place in the adjoining members.

It should be noted here, that this "flexible member" can only be placed where the stress is tensile. However, excepting the arch, which is in compression only, bridges of all classes contain tension members.

The foregoing discussion shows that the resistance to tension is the most important property of steel. By taking advantage of this property, in planning a great work, a technical and economical benefit may be realized, while, otherwise, work is done under unfavorable conditions.

The Flexible Member.—In order to make the action easily understood, take any construction, such as a derrick, Fig. 78, for example, composed of a vertical post, *D C;* an inclined boom, *A C;* an inclined tie, *E D;* a vertical hanger, *A B,* which carries an unknown load, *B;* and, finally, a flexible member, *A D,* acted upon only by a tension stress denoted by t_1; the whole forming an articulated system lying in the same plane.

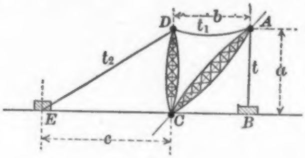

FIG. 78.

After t_1 has been determined, it is known that the tension, t, in *A B* is

$$t = t_1 \frac{a}{b}$$

the tension in *D E* is

$$t_2 = t_1 \sqrt{1 + \frac{a^2}{c^2}}$$

Mr. Arnodin.

the compression, c_1, in $A\ C$ becomes

$$c_1 = t_1 \sqrt{1 + \frac{a^2}{b^2}}$$

the compression, c_2, in $D\ C$ becomes

$$c_2 = t_1 \frac{a}{c}.$$

The stresses may also be found graphically, as shown in Fig. 79.

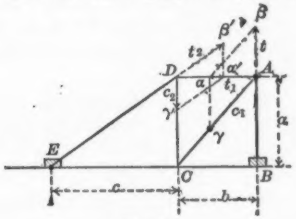

FIG. 79.

The problem is now to find the value of t_1.

It was to solve this problem, while making experiments for the construction of the ferry bridge at Rouen, that the writer conceived the "Tension Meter," the principle of which, under the name of "Test Cable," he explained before the Société des Ingénieurs Civils de France,* which principle is herewith reviewed.

The Tension Meter.—Since t_1 (Fig. 78) is in tension, a flexible cable may be used in its place; and it is perfectly reasonable to use it because, of all material at the disposal of the engineer, the cable, consisting of (nickel or carbon) steel wires, offers, with a minimum of weight, a maximum of resistance and safety. Therefore, this system involves a structural advantage rather than a disadvantage.

As the coefficient of elasticity of the steel in cables is very nearly the same as that in eye-bars, and as the variation in the curve of the cable within the limits of the variation of the load affects the total elongation very slightly, the increase in the flexibility of the structure will in no case result in a serious disadvantage.

t_1 being a cable, will assume, within its free length, the curve of a catenary under the twofold effect of its own weight and its tension. No human power and no secondary stress can oppose this natural law, which acts upon all the particles of the flexible member.

Suppose a longitudinal, and extremely thin, element of the cable to be separate from the entire free length of the member. To fix the idea conceive this elementary wire to have a cross-section of 1 sq. mm. The density, the flexure, and the tension of this wire, per unit section,

*At the meeting of February 20th, 1903.

Mr. Arnodin. are the same as those of the whole member. Therefore, it is only necessary to apply a dynamometer to this wire in order to observe the strain on the member; that is to say, it will show the actual stress, which includes not only the main stresses, but also any non-analyzable secondary stresses which may occur. In short, this elementary wire, conceived as separated from the remainder of the member, will not be the indicator of any theoretical deductions, but will show directly and accurately the actual condition of the member.

However, in practice, one cannot realize the conception of the isolated cable element, hence this difficulty must now be overcome.

This is accomplished simply and easily by adding, at the side of the flexible member, a very thin wire.

If this wire is of the same specific gravity, and if its free curve is the same as that of the member, then its tension must necessarily be that of the whole member. The amount of this tension, indicated by a dynamometer, will certainly throw light on the actual stresses.

Strictly speaking, the addition of the tension meter wire decreases slightly the stress of the member, since it takes its portion of the total stress; however, the error is a negligible quantity, and only has a value equal to $\frac{s}{S}$, in which S is the total section of the flexible member, and s the section of the wire of the tension meter.

In practice, the Arnodin tension meter consists of a wire, the cross-section of which is 1 sq. mm. It is of the same specific gravity as the main cable member, and at one end there is a light dynamometer, which in turn is attached to a regulating screw, with which the wire is adjusted to the same curve as the main cable in its unsupported length.

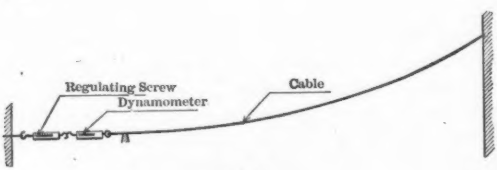

FIG. 80.

It should be noted that the greater the unsupported length of the cable, the greater is its curve, and the easier and more precise will be the observations. The flexible member, therefore, should be of great length, and for this reason bridges of large span are more favorable for this installation than those of smaller span. In such bridges, also, the observations are most valuable.

Further, the flexible member should always be either horizontal or diagonal, never vertical, for the method is based on the curve of the cable, which does not occur in vertical tension members.

Suspension bridges carried by cables of flexible wires, are, by their very nature, admirably adapted to this kind of testing, for in such bridges there can always be found a free inclined length of cable—the anchor span, for example—in which the tension meter may be placed; and it will be difficult to understand, once the system has become known, how engineers who are responsible for these great structures can afford to deprive themselves of this experimental control over the stresses actually carried by the cables. Mr. Arnodin.

In cantilever bridges, Fig. 81, such as the Forth, the Quebec, the Blackwell's Island, etc., nothing is simpler than to make of cables either of the members, A B or A C, which, being always in tension,

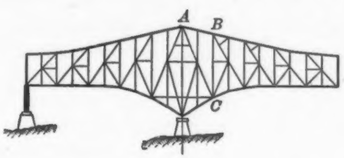

FIG. 81.

can be used as the "flexible member," so that its stress may be tested by the tension meter, both during construction, and later, during maintenance; and, from the observations, the stresses in the other members may be deduced.

It may be stated that if this precaution had been taken, there would not have been cause to deplore the Quebec disaster, nor the embarrassing situation of the Blackwell's Island Bridge; for, during the course of erection, the indications of the tension meter would have given warning of the errors in the computations, and in the assumptions upon which the design was based.

But even when a structure has been designed and erected with the greatest possible care, it would still be of benefit to be able to make sure whether or not it behaves according to the design, and whether, subsequently, dynamic strains or secondary effects do not modify the computed stresses.

In the writer's opinion, it would be very satisfying to be able, by the use of the tension meter and its gauge, to control the stresses of the Brooklyn Bridge when the traffic is at its maximum, or in the Forth Bridge, under a passing train, or in any other bridge of lesser span.

The French engineer, M. Leinekugel Le Cocq, who is a specialist in work of this class, has mentioned the remarkable results which he obtained by the aid of the tension meter, while in charge of the testing of the ferry bridges at Marseilles and at Brest,* which results he wisely compared with the results of the theoretical computations.

* *Le Génie Civil*, February 24th, and March 4th, 1906, and October 31st, 1908.

Mr. Arnodin. It ought to be realized that the extension of this system to all large structures would greatly promote that progress which engineers desire, for it would indicate by exact methods where the computations, or the theoretical hypotheses, were at fault, would lead to their rectification and would also clear up some disputed questions.

It would insure for the future a higher degree of safety in the execution of structures, a more judicious use of material, a better economy, and, above all, a greater confidence and sense of security on the part of the public.

Rigid Compression or Tension Members.—Many engineers have been on the lookout for a method of observing the actual strains, but not having at their disposal the "flexible member" and its measuring apparatus, because it was not known to them, they have attempted to observe the strain on rigid compression or tension members. In such a case one encounters a difficulty in principle, namely, the fact that the manifestation of strain is made possible only by the elastic property of the material, which is generally assumed to take a strain of $\frac{1}{20\,000}$ of its length for every kilogramme per square millimeter ($\frac{1}{29\,000\,000}$ per pound per square inch), tension or compression.

It follows that every error of observation, and every secondary consideration which affects the measuring apparatus will be multiplied by 20 000, and this might often lead to grave mistakes.

Again, the accepted value of $\frac{1}{20\,000}$ per kilogramme for the elastic elongation is not uniform. It varies with the temperature of rolling, with the chemical composition of the steel, and with many other unavoidable causes. It may thus happen that, of the many members in a bridge, those on which the tests are made possess an elastic coefficient other than the usual value.

Moreover, this method will only give the stress due to the load variation, and not the total stress. Thus it may happen that the member under observation is already strained so highly that it ought not to receive an additional stress due to an increase of load, and this additional stress is the only one which can be measured. By this method of observation, no real assurance of the degree of safety of the structure is obtained.

Very ingenious apparatus have been invented for such measurements. The best of these are the "Manet", and the gauges by Professor Rabut, of France, which make it possible, according to the picturesque words of the inventor, to "enquire of the bridge how it is getting on."

It follows, from the foregoing, that the engineer should not lose sight of his bridge as soon as he has completed its construction, but, like the good teacher who follows the life of his pupil, he ought to keep track of its career, observe its qualities and its defects, and try to remedy the latter, or at least deduce therefrom lessons which will help him in future structures.

As to "Removability."—The science of conserving the life of the structure has not been exhausted in the preceding remarks. Mr. Arnodin.

In the case of small bridges, involving a low cost of reconstruction, durability is perhaps not so important a consideration, for when the life of the structure is at an end, it is relatively a simple matter to erect in its place a new bridge.

But, in the case of structures of such great spans as have been considered herein—and in the future the spans may be greater still—which absorb millions of the public funds, it is necessary to be more far-sighted, and to provide at the very outset for a much longer life than is ordinarily assigned to metallic structures.

Up to the present there is not much definite information regarding the durability which a metal structure in the free air ought to possess. It is principally dependent on the degree of care in maintenance, and many of the best engineers believe that metallic bridges as now constructed will not endure for one hundred years; in fact, there are a number of bridges which should be renewed after having reached the age of thirty years.

This fact led the writer to introduce into suspension bridges, the principle of "Removability" [*l'amovibilité*], that is, to arrange the structure, at the outset, so as to be able in the future to replace the several parts, without affecting unfavorably the resistance of the whole, and without interrupting even temporarily the regular traffic on the bridge.

Accordingly, the structure may be made to last indefinitely by replacing individually, one at a time, as may be required, each of its members, just as a railroad track can be made to exist as long as the ground that supports it, by continually renewing the worn-out rails.

The principle of "Removability" is not yet applicable to all classes of bridges. It is particularly adapted for tension members, and, like the latter, for the "flexible member."

Danger of Inaccessible Parts.—The numerous examinations of old bridges made by the writer at the beginning of his practice, have led him to the conclusion that the greater part of the failures due to deterioration result from parts inaccessible to inspection or maintenance. Accordingly, he has spent thirty years of his career in making war upon the inaccessible parts of bridges, notably upon anchorages of suspension bridges, which, being generally in low and damp places, are most exposed to deterioration.

It is necessary, therefore, that the members of the anchorage should be entirely accessible, so that the engineer, by frequent inspection and by proper maintenance, may be enabled to repair the deterioration in time, and to see the trouble before a disaster unexpectedly reveals it.

Conclusions.—From the preceding considerations the writer reaches the following **conclusions:**

Mr. Arnodin. First.—That the use of nickel steel may be advantageously recommended for the tension members of bridges.

Second.—That its superiority over carbon steel should be utilized for the increase of safety rather than for the diminution of section.

Third.—It appears that engineers in the United States have been too daring in their specifications for allowable maximum stresses.

Fourth.—That it is advantageous, in the study of design, to devise forms of articulation which will utilize the tensile resistance of the metal, because tension eliminates flexure and a number of secondary stresses.

Fifth.—That the arrangement of certain tension members should be such as to form flexible members, which, by the aid of the tension meter, will enable the precise observation of the actual stresses.

Sixth.—It would be advantageous to have a bureau of control to test metallic bridges in order to make sure that the specified stresses are not exceeded, and that the deterioration of the metal does not affect their safety.

Seventh.—That no part of a bridge should be inaccessible to inspection and maintenance.

Eighth.—That those principal members, upon which the safety of the bridge depends, should, as far as possible, possess the advantage of "removability."

Regard for these conditions will add to the safety of bridges, and will greatly increase the confidence in the science of the engineer, and in the structures which he executes, no matter how great the apparent boldness of their design may be.

Mr. Worsdell. WILSON WORSDELL, ESQ.* (by letter).—The following particulars, relating to a nickel-steel fire-box which is being built for the North Eastern Railway, may be of some interest. The composition of this steel is as follows:

Nickel	16.3	per cent.
Carbon	0.54	"
Manganese	2.87	"
Phosphorus	0.027	"
Sulphur	0.032	"
Silicon	0.378	"

This steel gave a maximum ultimate stress of 40 tons per sq. in., with an elongation of 50% in 3 in., whereas an ordinary mild-steel sample of similar dimensions gave 29.6 tons per sq. in., with an elongation of 35% in 3 in. A strip of the nickel steel, ¼ in. thick, was bent double, cold, without cracking. When flanging the fire-box, the plates were annealed by heating them to redness and plunging them in water, the carbon having no hardening effect, due to the presence of the nickel.

* Chf. Mech. Engr., North Eastern Ry., Great Britain.

Although the steel showed such ductility under test, it was most difficult to machine and press. This was probably due to the manganese. Mr. Worsdell.

The writer was under the impression that the addition of nickel did not make steel any more difficult to machine, and that this had been demonstrated by Hadfield, although some American manufacturers consider that the nickel is responsible for this difficulty.

WILLIAM F. PETTIGREW, ESQ. (by letter).—The writer, having charge of the maintenance of several bridges and viaducts, has often thought that nickel steel could be used for such structures, particularly with reference to decreasing the corrosion between wind and water, that is, where the material is at times covered with water and then exposed to the atmosphere. Mr. Pettigrew.

In two of these viaducts the columns are of cast iron braced by wrought-iron tie-rods; in one case 1½-in. and, in the other, 2-in. rods are used. A space of 7½ ft., embracing parts of these rods, is exposed to water and the atmosphere intermittently, and at certain places there is corrosion. The writer has often thought that, if these tie-rods were of nickel steel, this difficulty of corrosion would be considerably overcome, but the trouble has been to obtain nickel-steel bars, although application has been made to many manufacturers. This is confirmed by Dr. Waddell, as he states that up to the present time the only nickel-steel bars manufactured, within his knowledge, have been made by the American Bridge Company. The results he has obtained in his tests are certainly most satisfactory, and will be a great boon to engineers in the future.

In reference to the viaducts under the writer's control, the corrosion and salt tests are very interesting, and prove that in both cases the loss with nickel steel is much less than with ordinary carbon steel.

J. A. L. WADDELL, M. AM. SOC. C. E. (by letter).—In beginning this résumé the writer desires to tender his hearty thanks to all who have been so courteous as to discuss his paper. It is true that he had hoped for a much larger discussion, especially from certain prominent American bridge engineers whose opinions on the various matters treated in the paper would certainly carry great weight; but he recognizes that the paper is difficult to discuss, since it treats of a subject which is almost entirely new in engineering. The writer feels especially indebted to the various foreign engineers who have done him the honor to discuss the paper, and begs to tender them herewith the assurance of his deep appreciation of their consideration and courtesy. Mr. Waddell.

In some of the discussions certain side issues of much value are treated; but the writer deems it best, in order not to extend the discussion unnecessarily, to omit practically all reference to these in his résumé.

Mr. Waddell. Again, he takes cognizance herein of only one-third of the discussions. By so doing he does not by any means intend to imply that the others are unworthy of notice, but brevity demands that he refer only to those points upon which he differs materially from the various writers. Their discussions will be treated hereinafter in alphabetical order.

In bridge designing there are certain restrictions governing minimum sections and minimum thicknesses of metal, which all thoroughly-written bridge specifications recognize, consequently Mr. Arnodin's criticism of nickel-steel bridges on account of possible ultra-small sections and ultra-thinness of webs will not hold. Moreover, in computing the weights of metal and the comparative costs of spans given in the diagrams, the writer took due account of these restrictions in designing.

Mr. Arnodin's objection to nickel steel because of its tendency to increase vibration due to reduction of total weight of structure will apply properly to short, light spans, such as those of county bridges; but in modern railway bridge designing the live loads are so great that the main members and connecting details of spans of even ordinary length become bulky and clumsy. The effect of the use of nickel steel would be to reduce this objectionable feature of modern bridge designing.

Mr. Arnodin has a wrong notion in mind when he states that the use of nickel steel will reduce the safety and durability of a bridge. The writer's whole economic investigation is based upon keeping the safety, strength, and durability of both carbon-steel and nickel-steel bridges alike, and ascertaining the relative costs of the two kinds of superstructure. The specifications of the paper were intended to be drawn so as to keep these attributes as nearly identical as practicable, and the writer believes that they will accomplish that purpose, except, perhaps, that the durability of the nickel-steel structures will be greater than that of the carbon-steel structures, because of the alloy's superior resistance to corrosion.

Mr. Arnodin is in error when he states that the writer advises "mixed structures in which the compression members are to be of carbon steel and the tension members of nickel steel," for in his "mixed metal" bridges he suggests using carbon steel only in those parts where the adoption of the stronger metal would effect no good purpose or economy. Nickel steel is just as fit for compression members as carbon steel, provided proper consideration be given to the designing and detailing. Compression members which either fail or do not develop the proper strength are deficient because they lack proper design and detailing, and not because they are compression members ***per se***.

Mr. Arnodin's statement that the stresses allowed on nickel steel in the Blackwell's Island Bridge are too high is, in the writer's judgment, correct; because the greatest intensity for live load, impact allowance load, and dead load should not have exceeded 30 000 lb. on eye-bars, while 39 000 lb. was adopted. Even the impossible combination of greatest assumed live load, impact allowance, dead load, and wind load should not stress the eye-bars higher than 37 500 lb. per sq. in. Mr. Waddell.

It would not be scientific engineering to lower the intensities of working stresses in a bridge in anticipation of a possible augmentation of dead load during manufacture and construction, because every bridge should be designed complete in every detail before the drawings are sent to the shops, and it is the duty of every bridge designer to check the dead load of every structure from the completed drawings before letting the latter pass out of his possession and control.

It is not the writer's intention to discuss Mr. Arnodin's "Tension Meter," but he cannot see how such an apparatus placed at the top of the tower in the Quebec cantilever bridge could have indicated an impending failure in one of the bottom chords.

The writer does not agree with Mr. Arnodin's suggestion that bridges should be designed so that all their parts may be removed and replaced as they wear out, for this would involve great lack of rigidity as well as a multitude of constructive complications; but he would prefer to utilize the principle adopted by the designer of the famous structure known as the "Deacon's One-Horse Shay," in which all parts were equally strong, gave good service without repairs for many years, and then suddenly went to pieces because of the uniform deterioration and simultaneous collapse of all its members.

Mr. Carpenter's dread of mixing the steels improperly, in a bridge built of both nickel and carbon steels, is unnecessary. It was not the writer, but his inspector, Mr. Saunders, who on page 248 said that "extreme precautions were taken at every step to keep the two steels separate." These precautions were adopted by the inspector so as not to spoil the results of the tests of apparently identical columns made of the two kinds of steel. In any well-organized shop the marking system will make it certain that the right kind of steel is used in each place in mixed steel bridges.

If one were dealing with the usual English ratios of depth of truss to length of span, viz., about 1 to 12, instead of the customary American ratios of about 1 to 6, or 1 to 7, he might have cause to worry over the increase in deflection due to using nickel steel; but the deflections of modern American bridges are so slight that this matter is hardly worthy of consideration, in so far as they affect the camber and the track.

Mr. Waddell. There is something in Mr. Carpenter's statement that the greater deflections of nickel-steel structures will augment the secondary stresses as compared with the equivalent carbon-steel structures; but, on the other hand, the smaller dimensions of the nickel-steel members will have a contrary effect. Much has yet to be learned about the gravity and extent of secondary stresses in bridges, and how best to avoid or provide for them.

Mr. Carpenter evidently rivets his stringers together continuously from end to end of span, even in long-span structures; but the writer's practice is to insert occasional expansion pockets in his floor-systems so as to prevent the stringers from doing some of the work of the truss chords.

As for the greater distortion of nickel steel affecting the adherence of the paint—does not such a thought involve the stretching of one's imaginative faculty beyond the elastic limit?

If there is any other alloy of steel that will give as good results as nickel steel at less cost, the Profession ought to know it. The writer did look into vanadium steel, but discovered that it is altogether too expensive for bridgework. If vanadium could be furnished at any reasonable cost, vanadium steel might compete successfully with nickel steel.

As for the accuracy of the Phœnix testing machine, the writer, when writing the paper assumed that the numerous testing machines used in the investigation by his various inspectors gave correct records. It would, of course, be desirable to have this doubt about the Phœnix records of column resistances removed; but, unfortunately, the writer's time is so taken up with important professional work that it is impracticable for him to make the investigation, especially as the time limit set for the completion of this résumé of discussions is nearly reached.

In any case the variation of the machine would not affect very greatly the ratio of strengths of nickel-steel and carbon-steel struts, even if the actual strengths recorded were wrong; and it must be remembered that the specifications for nickel-steel bridges and the comparisons of cost made by their use were based on the comparative strengths of nickel steel and carbon steel.

It seems to be a slur upon the Engineering Profession that there should be any doubt about the correctness of results from such a large and important testing machine as the one at Phœnixville; and it is to be hoped that when the great machine upon which Mr. Emory is now working is completed, engineers will have at their disposal an apparatus which may be relied upon absolutely.

The writer investigated the use of ferruginous nickel, mentioned by Mr. Fowler, but found that, unfortunately, it contains so much copper as to prohibit its use for manufacturing nickel steel without first separating the nickel. Mr. Waddell.

The writer does not believe that the acid open-hearth process will produce any better high-grade steel for bridges than the basic open-hearth process as now operated; in fact, in his opinion, the latter is preferable.

Mr. Hallsted's criticism of the increase in rivet diameter not being applicable to the flanges of channels is correct; but the writer would state that, unless American manufacturers decide to roll channels deeper than 15 in., there will be but little use for these sections in nickel steel. For some years, at least, nickel steel for bridge building will be confined to long spans, where channels of obtainable dimensions are inadmissible.

In plotting his economic curves, the writer did not forget the decrease in economy due to the increased sectional areas made necessary by the larger rivet holes in nickel steel.

Mr. Hallsted's various anticipations of increased cost are mainly those which will exist in the transition stage, while carbon steel is being gradually supplanted by nickel steel. They would soon be forgotten, as were the similar anticipated troubles when carbon steel was about to replace wrought iron for bridgework.

Mr. Hallsted has made an error in quoting the writer's "De Pontibus" formula for top chords. He assumes it to be $16\,000 - 70\,\frac{l}{r}$ and compares that with the nickel-steel formula of $30\,000 - 120\,\frac{l}{r}$, while it should have been $18\,000 - 70\,\frac{l}{r}$. This explains why he thought he found the writer in error.

Mr. Lindenthal is mistaken when he says that a large demand for nickel would probably raise its price. On the contrary, such a demand would lower the price materially, because the supply of nickel ore already in sight is immense, and an increased demand would certainly result in improved methods of extracting the metal from the ore.

The writer does not agree with Mr. Lindenthal when he says that the use of high steels in bridge construction is justified only in very long spans, because (as previously stated) modern live loads have become so great that in spans of moderate length, especially for railroad bridges carrying more than one track, the main members are becoming too bulky and the connecting details in riveted structures too clumsy.

Mr. Waddell. The deflections of modern, riveted bridges which are correctly designed, and in which the camber is halved by proper adjustment of ties, are of such minor importance (excepting the effect of secondary stresses) that they may be forgotten, whether the material of the superstructure be carbon steel or nickel steel.

The writer agrees with Mr. Lindenthal that "to use high steel [including nickel steel] in a light structure, for the sake of low first cost, is not true economy," and it was on this account that he suggested that the alloy should not be adopted for highway bridges, unless these be of an unusually heavy character.

The writer does not agree with Mr. Perry that experiments should be made upon the effect of continued vibration on nickel steel, because it is generally conceded to-day by the leading bridge engineers that the effect of repeated stress on bridge members, provided the elastic limit is not exceeded, is absolutely nil. The idea of deterioration of steel bridges by vibration or by fatigue from repetition of stress is a bugbear that has, fortunately, been relegated to the past, and the oppressive nightmare which it involved no longer disturbs the slumbers of the bridge engineer.

In speaking of the cost of fabricating nickel steel, Mr. Prichard states that the writer's estimate is too favorable to the alloy. The writer, in proof of the correctness of his figures, begs to quote the following letters from two high authorities upon such questions:

"September 3, 1908.

"Mr. J. A. L. Waddell,
"608-11 New Nelson Bldg.,
"Kansas City, Mo.

"Dear Sir: Referring to your letter of the 25th ultimo relative to our experience with nickel steel for bridge work, we beg to state that in so far as the ordinary operations of ship building are concerned we find that the cost of working nickel steel compares very favorably with that of mild steel. Our experiments were not made in such detail as to make a discussion by us profitable.

"We have, however, worked nickel steel in government vessels for a considerable period, and have no difficulty whatever with it in the usual shop manipulations. The cost of drilling is slightly greater than that of medium steel.

"We feel confident that the material you propose may be used in bridge work to great advantage in lessening the weight without materially increasing the cost.

"Very truly yours,
"The William Cramp & Sons Ship & Engine Building Company,
"W. A. Dobson,
"Naval Architect."

"PITTSBURG, PA., DEC. 26TH, 1907. Mr. Waddell.

"WADDELL & HARRINGTON,
"MR. JOHN LYLE HARRINGTON,
"KANSAS CITY, MO.

"MY DEAR MR. HARRINGTON: Some time ago you wrote us in regard to the increased cost of the shop work on nickel steel over carbon steel.

"Since receipt of your letter we have made some experiments, and as near as we can tell from the experiments made, the additional cost of shop work will be about 10% higher for nickel steel than for ordinary steel.

"We will be pleased to receive your opinion as to the probable difference in cost in working this material.

"Yours truly,
"MCCLINTIC-MARSHALL CONSTRUCTION CO.,
"H. H. MCCLINTIC,
"V. P. & Gen. Mgr."

Mr. Prichard, like some of the other engineers who have discussed the paper, fears that trouble may be encountered in the shops by the difficulty in distinguishing between nickel steel and carbon steel. This anticipation is needless, for the instant any tooling or shop manipulation of any kind is started the nickel steel will declare itself; moreover, the exterior surface of the alloy can generally be relied on to indicate its character.

Mr. Prichard also fears that, in the future, inspecting engineers will not be able to determine whether a bridge was built of carbon steel or nickel steel. Again, the anticipation is unwarranted, for a few applications of a file would tell the tale; moreover, all railroad companies now-a-days keep office records of their structures in which the characteristics of the metal are given, as do also the manufacturers of such structures. In the examinations of old railroad bridges which the writer is constantly making, he experiences very little difficulty in ascertaining what structures or parts of structures were built of steel and what of wrought iron.

Mr. Prichard states that:

"Members in compression, in addition to resisting the effort of the load to crush them, have to resist its tendency to buckle and wrinkle them, and the resistance to these tendencies is about the same for steel of all grades."

Had Mr. Prichard read carefully the writer's description of his experiments on full-sized columns, he would not have made such a mis-statement. Nickel steel resists "buckling and wrinkling" in columns, up to working limits of length, far better than carbon steel.

The writer confesses that his specifications for nickel-steel eye-bars are not based upon complete experiments, and he regrets deeply that he was unable to procure proper eye-bar steel for testing; nevertheless, he feels confident that, when a comprehensive, scientific study is made

Mr. Waddell. of the best composition and method of manufacture for nickel-steel eye-bars, it will be found that his specifications for these members will not be far astray.

About the first investigations that would have to be made before using nickel steel in any great bridge are the composition of eye-bar steel and the proper method of annealing nickel-steel eye-bars. There is yet much to be learned about both these matters, and it is to be hoped that the engineers of the Quebec Bridge will soon inaugurate such investigations.

Mr. Prichard terms the unit stresses of the "De Pontibus" specifications "high." He seems to forget that they are for equivalent static loads. As such, they are not high, compared with the general practice of the leading American designers and manufacturers of medium-steel bridges.

The reasons why the writer advocates the basic open-hearth steel for the manufacture of nickel steel for bridges are:

First.—This process is used almost exclusively in the United States.

Second.—The basic process permits of cutting down the percentage of phosphorus to an exceedingly small amount, while the acid process does not; and experience has shown that phosphorus is especially objectionable in nickel steel.

Mr. Ross is right in stating that the corrosion experiments should be carried further. The writer would suggest subjecting both nickel steel and carbon steel to the deteriorating influences of the salt air and salt water of the Gulf of Mexico, where he has seen railway rails corrode in a few years to such an extent as to be almost unrecognizable as rails.

The writer said nothing about the use of nickel steel for reinforcing concrete because he does not believe it is well fitted for the purpose. Ordinary medium steel is strong enough in all conscience—and possibly too strong, considering the adhesive strength of the mortar to the metal. The use of nickel steel would reduce the ratio of strength of adhesion to strength of metal, and would thus intensify the most serious reason for apprehending that the life of reinforced concrete structures is limited.

In reference to the use of nickel-steel rivets in nickel-steel plates and carbon-steel rivets in carbon-steel plates, to which Mr. Sparrow calls attention, the writer would state that his remark applied mainly to test specimens; for it is obvious that in a bridge no harm could be done by using nickel-steel rivets to connect carbon-steel stay-plates or lacing bars to nickel-steel main members; but, in lateral struts, having both the sections and the details built entirely of carbon steel, it would, of course, be preferable to use carbon-steel rivets.

Mr. Waddell.

In conclusion, there are certain points to which the writer desires to call attention.

In the paper as printed herein much more attention is given to mixed-steel bridges than to bridges built entirely of nickel steel; but in the original paper, which is on file in the Society's Library, both classes of structure receive the same consideration. When it became necessary, on account of economy of space and cost, to reduce the original paper to much smaller dimensions, the writer and the Committee to whom the paper was referred concluded that, as the transition from carbon-steel bridges to nickel-steel bridges would probably take place through the medium of "mixed-steel" bridges, it would be best to omit all the comparative cost diagrams relating to bridges built of nickel steel throughout. It is possible that in some future publication the writer will print these omitted diagrams; for, if nickel steel ever does begin to replace carbon steel for bridgework, the time will certainly come when for various good reasons carbon steel will not be mixed with nickel steel in the same span.

The next step to take in order to determine the desirability of the use of nickel steel for bridges is for some engineer, who has a large bridge to build, to make preliminary plans and specifications for superstructures of both nickel steel and carbon steel and call for bids thereon, thus ascertaining with certainty the present economics of the two metals; then, if nickel steel shows a great advantage in price, it will be necessary to make some further tests, mainly of eye-bars and full-sized columns, so as to settle finally the proper intensities of working stresses for such members in advance of the preparation of the working plans.

The adoption of nickel steel is really being forced upon the Profession by the call for bridges of exceedingly long span to support large live loads. In the stiffening trusses of the great Manhattan Bridge it proved necessary to adopt nickel steel to keep the weight of the structure down to reasonable limits; and, in the writer's opinion, such a course will be found obligatory in the reconstruction of the long cantilever bridge at Quebec. The main span of this structure, 1 800 ft., is very close to the greatest practicable limit for cantilever bridges of carbon steel. The writer's calculations show that, at present prices of carbon steel and nickel steel, a saving of from 25 to 30% in the cost of the superstructure could be effected by adopting nickel steel in those parts where its use would be advantageous or economic. Consequently, the committee of engineers in charge of the work will no doubt carefully consider the advantages of nickel steel before deciding finally upon the character of the new superstructure.

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TRANSACTIONS

Paper No. 1104

THE IMPROVEMENT OF THE OHIO RIVER.*

By William L. Sibert, M. Am. Soc. C. E.†

With Discussion by Messrs. Theron M. Ripley, and William L. Sibert.

The recent completion of Locks and Dams Nos. 2, 3, 4, and 5, Ohio River, creating, in connection with Locks Nos. 1 and 6, a navigable depth of 9 ft. from Pittsburg to Beaver, Pa., a distance of about 30 miles, and constituting the first completed section of the 9-ft. project, caused the writer to think that a review of the various engineering projects proposed for the improvement of this river, might be of interest to the Profession.

The writer had charge of the construction of Locks Nos. 2, 3, 4, 5, and 6, from 1903 to the spring of 1907. Captain E. N. Johnston, Corps of Engineers, U. S. A., one of his assistants at that time, aided materially in preparing this paper.

Physical Characteristics.

The Ohio River is formed, at Pittsburg, by the junction of the Allegheny and the Monongahela Rivers. Its total length, all of which is navigable, is 967 miles. The navigable length of its tributaries is about 3 000 miles. The drainage area above Pittsburg, including the valleys of the Allegheny and the Monongahela, is about 19 000 sq.

* Presented at the meeting of December 16th, 1908.

† Major, Corps of Engineers, U. S. A.

miles, while the entire water-shed comprises an area of about 214 000 sq. miles, which is larger than the area drained by the Mississippi River above the mouth of the Missouri, and larger than the drainage area of any other river in the United States, except the Missouri and the Lower Mississippi. The total fall at low water from Pittsburg to Cairo is 426 ft. In the first 26 miles of the river, from Pittsburg to Beaver, the average slope is about 15 in. per mile; from Beaver to Wheeling, a distance of 64 miles, 9 in. per mile; from Wheeling to Louisville, a distance of 509 miles, 5½ in. per mile; while from Louisville to Cairo, the slope is only 4 in. per mile.

In its upper portion, the river is subject to frequent and rapid fluctuations of water surface.

In connection with the generally accepted theory that the destruction of the forests has increased the number and extent of floods, and, at the same time, decreased the low-water flow of the streams, the following statistics are cited:

Records of the river stages in Pittsburg for the 24 years preceding 1881, show the following:

River Stage.	Average Number of Days per Year
½ to 1 ft.	½ day.
1 to 2 ft.	34 days.
2 to 3 ft.	37 days.
3 to 4 ft.	47 days.
4 to 5 ft.	43½ days.
5 to 6 ft.	40 days.

Average number of days when river was below 6 ft., 214.
Average number of days when river was above 6 ft., 151.

For the 24 years succeeding 1881:

Average number of days when river was below 6 ft., 201.
Average number of days when river was above 6 ft., 164.

In Colonel Ellet's book, "The Ohio and Mississippi Rivers," the following record of lowest water depths on the bar at Wheeling is found:

Sept. 27, 1838	0 ft.	10½ in.	Sept. 19, 1845	2 ft.	2 in.
Sept. 6, 1841	1 "	0 "	Oct. 13, 1846	1 "	9 "
Aug. 17, 1843	1 "	8 "	Sept. 8, 1847	2 "	3 "
Sept. 23, 1844	1 "	1½ "	Sept. 18, 1848	1 "	11 "

The number of times that the Ohio River was above the danger stage at Pittsburg, Cincinnati, and Louisville, during the sixteen years preceding 1891 and the seventeen years following, is shown in Table 1:

TABLE 1.

Place.	Height of danger line above zero of gauge, in feet.	First period, 16 years, 1875-1890, inclusive. Times.	Second period, 17 years, 1891-1907, inclusive. Times.
Pittsburg, Pa.	22	18	26
Cincinnati, Ohio	50	11	11
Louisville, Ky	28	11	9

It is to be expected that the records at Pittsburg would show a greater number of river stages above 22 ft. in the years succeeding 1890 than in the preceding years, because of the material contraction of the river channels at that place due to encroachments of railroads and manufacturing plants. However, the conditions at Louisville and Cincinnati have changed so little that fair conclusions can be drawn from the records at those places.

While the highest gauge readings of which there is record in Pittsburg occurred March 15th, 1907, on which date a stage of 35.6 ft. was reached, it is thought that during the flood of 1832, which reached a height of 34.94 ft. at Pittsburg, the discharge of the river was probably equal to that of March 15th, 1907. A comparison of the old and present maps shows a marked contraction of the river channels near Pittsburg on account of slag dumped over the banks into the streams.

It seems to follow from these data that the extremes as to flood height are not materially influenced by forests, but that the frequency of medium size and possibly destructive floods may be increased by deforestation. These data further show that the deforestation of the Ohio water-shed has had practically no effect upon the extent of the low-water navigation seasons.

A theory has been advanced that the low-water stages of rivers are less dependent upon springs than upon summer rains, first on one tributary and then on another. This has been especially noted on the Monongahela River, which stream becomes exceedingly low during the summer and fall months, unless there be local rains, notwithstanding

a copious prior winter and spring rainy season. Summer rains, if the lands be denuded of forests, reach the river; whereas, if a forest exists, the rain is absorbed by the thirsty plant life, or evaporated.

The maximum recorded discharge of the Ohio River just below Pittsburg, is 439 000 cu. ft. per sec. The discharge at Davis Island Dam, just below Pittsburg, has been observed in recent years to be as small as 1 600 cu. ft. per sec. Captain Saunders gauged the Ohio River at Pittsburg in 1838, during the drought, and found the discharge to be 1 400 cu. ft. per sec. The quantity of water emptied into the Ohio by its tributaries during the dry season is very small, the low-water discharge of the Monongahela River being only 166 cu. ft. per sec. Although the low-water discharge of practically all the tributaries is too small, and the slopes of many of them are too steep to admit of an efficient improvement by regularization, the discharge is large enough to permit an extensive commerce to be carried on through a system of locks and dams, that on the Monongahela River being about 12 000 000 tons per year. It is not thought that such an extensive use of this or other streams of similar slope would be possible unless the same were canalized, this assuming the existence of sufficient water to make the needed open-river channel depths. The present system of dams with movable tops will create a navigable depth of about 11 ft.

Even during the dryest periods, an abundance of water exists in the Ohio for the maintenance of a 9-ft. depth in the pools of a slack-water system, as has been demonstrated at Davis Island Dam, built more than twenty years ago.

Usually, the low-water period includes July, August, September, October, and November. During the low-water season, the depth of water on the bars near Pittsburg is from 12 to 18 in.; sometimes for nearly three months the bar depths are less than 2 ft. The lack of navigable depth in the lower river is caused largely by the excessive width of the channel.

Table 2 shows the number of days, within the period named, that a 9-ft. stage or more existed at important points along the river.

The present project for improving the Ohio contemplates a navigable depth of 9 ft., which project, without changing any structures, could easily provide for 11-ft. navigation by dredging in the upper end of the pools. It will be observed from Table 2 that the number of days per year during which there is a natural stage of 9 ft. or more, in-

creases as the river is descended. From Pittsburg to Beaver, the mean number of days that a 9-ft. navigation is afforded is eighty-one. The Pittsburg District, including a radius of 50 miles around the city proper, is the most prolific freight-producing center in the world. The intermittent navigable stages in the river, aggregating only 81 days per year, has debarred this district from an extended use of the Ohio, except for the cheaper grades of freight, such as coal, and for that only where a long haul is necessary.

TABLE 2.

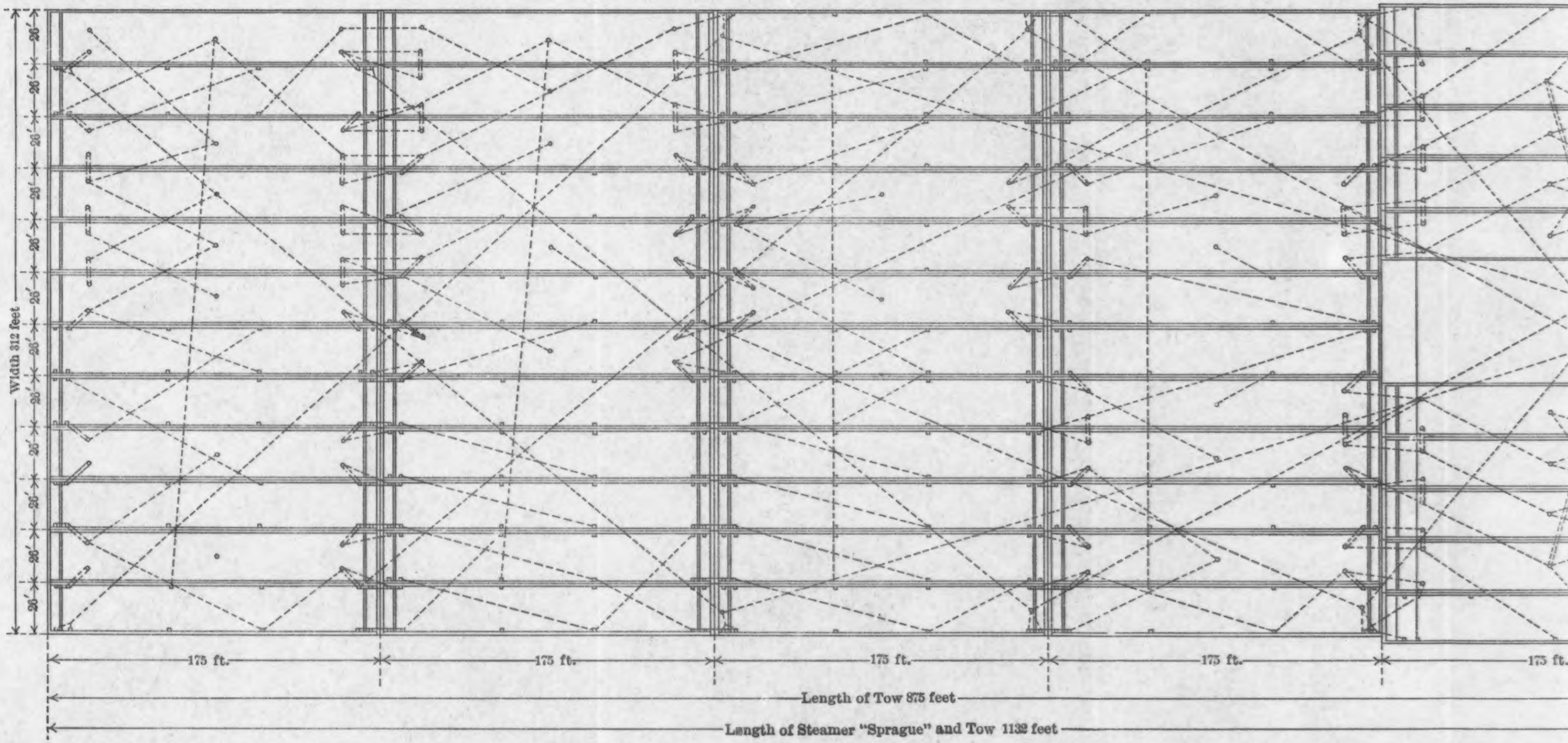
Place.	Miles.	Mean number of days at or above 9 ft. during the 10 years, 1895–1905.
Pittsburg, Pa.	0.0	81.0
Dam No. 6	28.5	117.0
Wheeling, W. Va	90.0	119.2
Parkersburg, W. Va.	183.5	140.5
Point Pleasant, W. Va.	263.4	150.3
Portsmouth, Ohio*	353.0	216.4
Cincinnati, Ohio	466.5	248.3
Madison, Ind.*	553.5	233.0
Louisville, Ky	599.0	97.5 Head of Falls.
Evansville, Ind	783.0	198.5
Paducah, Ky.*	919.5	203.6
Cairo, Ill	967.0	302.4

* Five years' records only.

During the periods that intervene between freshets, a great quantity of freight is kept in the harbor at Pittsburg awaiting a rise. The steamboats are tied up, much capital is idle, and great expense is incurred in watching the boats and barges, pumping them out, etc.

"In June, 1895, there were collected in the Pittsburg harbor, 1 200 000 tons of coal, loaded upon about 2 500 vessels awaiting water to move them down the Ohio River, the largest tonnage ever assembled in any harbor of the world at any one time. The rise did not come until November 27th. The cost of freight and vessels engaged in this service was estimated at $6 310 000. It cost $2 000 per day to keep the tonnage afloat, and $1 000 per day interest on the investment. Total, $3 000 per day. This tonnage was kept waiting in the Pittsburg harbor for water in the Ohio River an average time of five months, or 150 days, at a loss of $450 000, which is 5% of $9 000 000. This shows what one item of commerce lost in five months, because Western Pennsylvania did not have the economy of transportation that results from continuous water movement."*

* Address by Mr. John E. Shaw, before the Merchants' and Manufacturers' Association, at Pittsburg.


Width 312 feet
20'
175 ft.
175 ft.
175 ft.
175 ft.
175 ft.
Length of Tow 875 feet
Length of Steamer "Sprague" and Tow 1132 feet

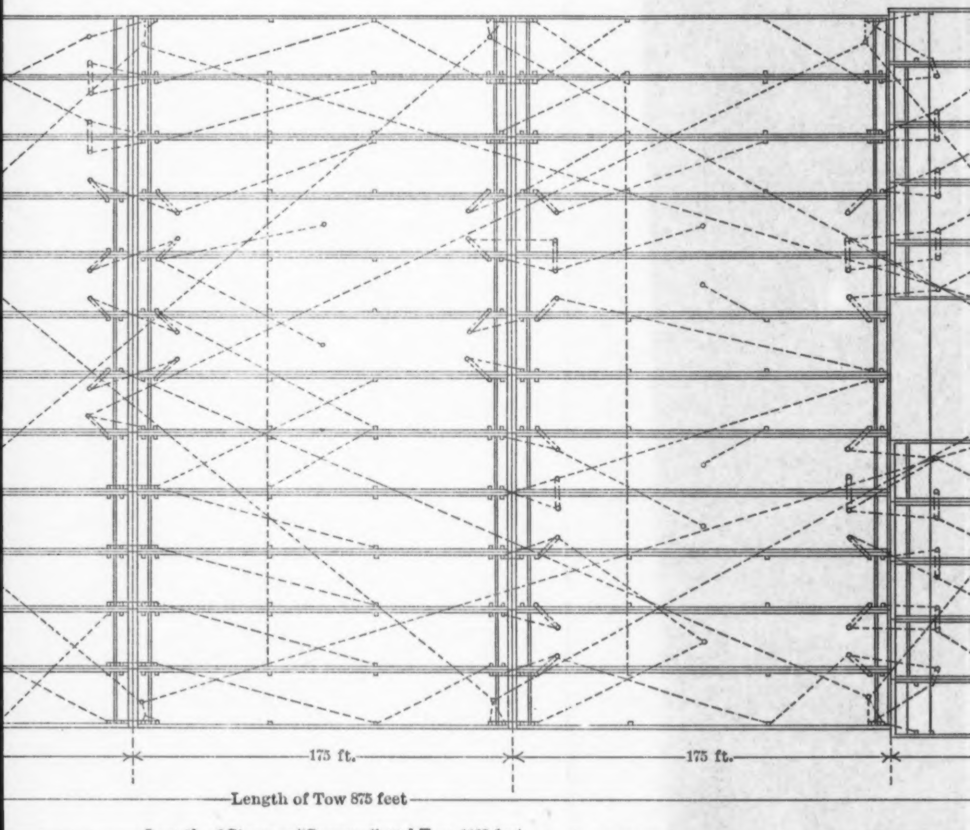
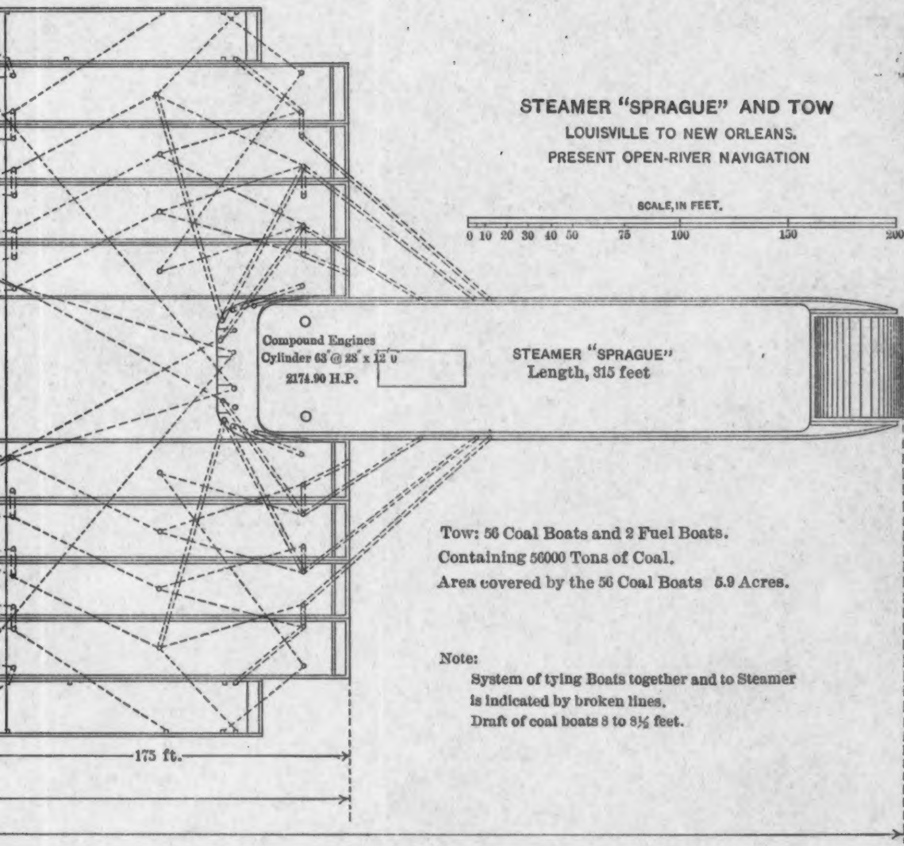

175 ft.
175 ft.
Length of Tow 875 feet
Length of Steamer "Sprague" and Tow 1132 feet

PLATE XX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXIII, No. 1104.
SIBERT ON
IMPROVEMENT OF THE OHIO RIVER.

STEAMER "SPRAGUE" AND TOW

LOUISVILLE TO NEW ORLEANS.
PRESENT OPEN-RIVER NAVIGATION



Tow: 56 Coal Boats and 2 Fuel Boats.
Containing 56000 Tons of Coal.
Area covered by the 56 Coal Boats 5.9 Acres.

Note:
System of tying Boats together and to Steamer is indicated by broken lines.
Draft of coal boats 8 to 8½ feet.

Boats and Commerce.

Boats suitable for navigating the Ohio River do not anchor, but are tied to the shore, and a sloping shore is needed so that they may be moved toward the bank as the water rises, thus keeping out of the stronger currents where drift would lodge against them and break them loose.

The short-lived freshets of the Upper Ohio have developed an unique system of towing on the Ohio and Mississippi Rivers. At times of freshets, freight is carried in great fleets made up of boats and barges. The boats are about 26 ft. wide by about 175 ft. long, and each carries approximately 1 000 tons. The barges are 26 ft. wide by about 130 ft. long, and each carries about 500 tons. As they leave the upper river, the fleets vary in size, containing from 10 to 20 boats or barges. These fleets are increased in size as they proceed down the river, and sometimes comprise from 50 to 60 boats of 1 000 tons each, the fleet covering 5 or 6 acres of water.

Plate XX shows the Steamer *Sprague* and a tow of coal barges.

When loaded, coal barges draw 6 ft., and coal boats from 8 to 9 ft., the former requiring a navigable depth of 8 ft., and the latter from 10 to 12 ft. The excess depth of from 2 to 3 ft. is needed, because the large fleets generally require a channel width of at least 300 ft. of the full depth of the deepest laden craft in the fleet, and to obtain this it is generally necessary to have a river stage at least 2 ft. higher than that necessary for a single boat. A slack-water depth of 9 ft. is at least equal to an 11-ft. open-river depth. With slack water, the pools are more than 9 ft. deep at all places except immediately below the locks. There being no currents to interfere, a little dredging allows fleets of full depth and width to pass into the deeper water and wider channels further down in the pools. The absence of currents greatly assists navigation in keeping in narrow channels.

The steamboats are of the stern-wheeled type. Experience has shown that these boats are exceedingly efficient for the work required. They are equipped with long balanced rudders against which the water from the wheel impinges with high velocity when the boats back, thus driving the stern in the desired direction whether the boat has headway or not. With this device, boats of 2 000 h. p. can successfully maneuver, down stream in swift water, fleets of from 40 000 to 60 000 tons burden.

In a report submitted to the Chief of Engineers, in 1870, the late W. Milnor Roberts, Past-President, Am. Soc. C. E., stated that:

"A system which will provide uninterrupted navigation will undoubtedly revolutionize the system of coal carriage. Less powerful steamers can then be used steadily all the year round, making regular trips, each boat taking fewer barges, but doing the same, or more business, with less capital invested at less risk, and without the expense due to idle boats and the expense of watching the great fleets while awaiting a freshet."

A system of locks and dams was referred to by Mr. Roberts in the paragraph quoted.

The records of freight cost under the present towing system, together with the estimated cost with a slack-watered Ohio, indicate that the export freight of the Ohio Valley, offered in barge-load lots, should be delivered at New Orleans at about $1 per ton.* Boats could then advantageously transport freight up and down stream. In an unimproved river, or in one improved by regularization, currents are often a serious handicap to up-stream navigation.

By adding to the foregoing cost a reasonable water rate from New Orleans to San Francisco, including a toll charge of 75 cents per ton through the Panama Canal, it appears that, with a slack-watered Ohio and a completed Panama Canal, freight should be transported from the Ohio-Mississippi water-shed to the Pacific Coast at from $5 to $8.50 per ton. With the present class rates by rail from Chicago or New York to San Francisco at from $19 to $32 per ton, and with commodity rates at about 4.3 mills per ton-mile, one would expect a large commerce, *via* the all-water route, between the people of the Ohio-Mississippi water-shed and those living between the Rockies and the Pacific Ocean. The Panama Canal will, of course, offer similar opportunities to the Atlantic Coast, both as to trade with the Pacific Coast and with China and Japan. In this connection the following is inserted:

"Recently 2 500 tons of steel rails were shipped from Pittsburg for the construction of the Prince Rupert and of the Grand Trunk Pacific System, by what, on its face, appears to be an exceedingly roundabout route. The rails were first sent to New York, where they were loaded on vessels going to the Orient by the Suez Canal Route. On reaching Kobe they will be transshipped and sent across the Pacific to their

* Executive Doc. 492, 60th Cong., 1st Sess., pp. 33 to 35.

PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
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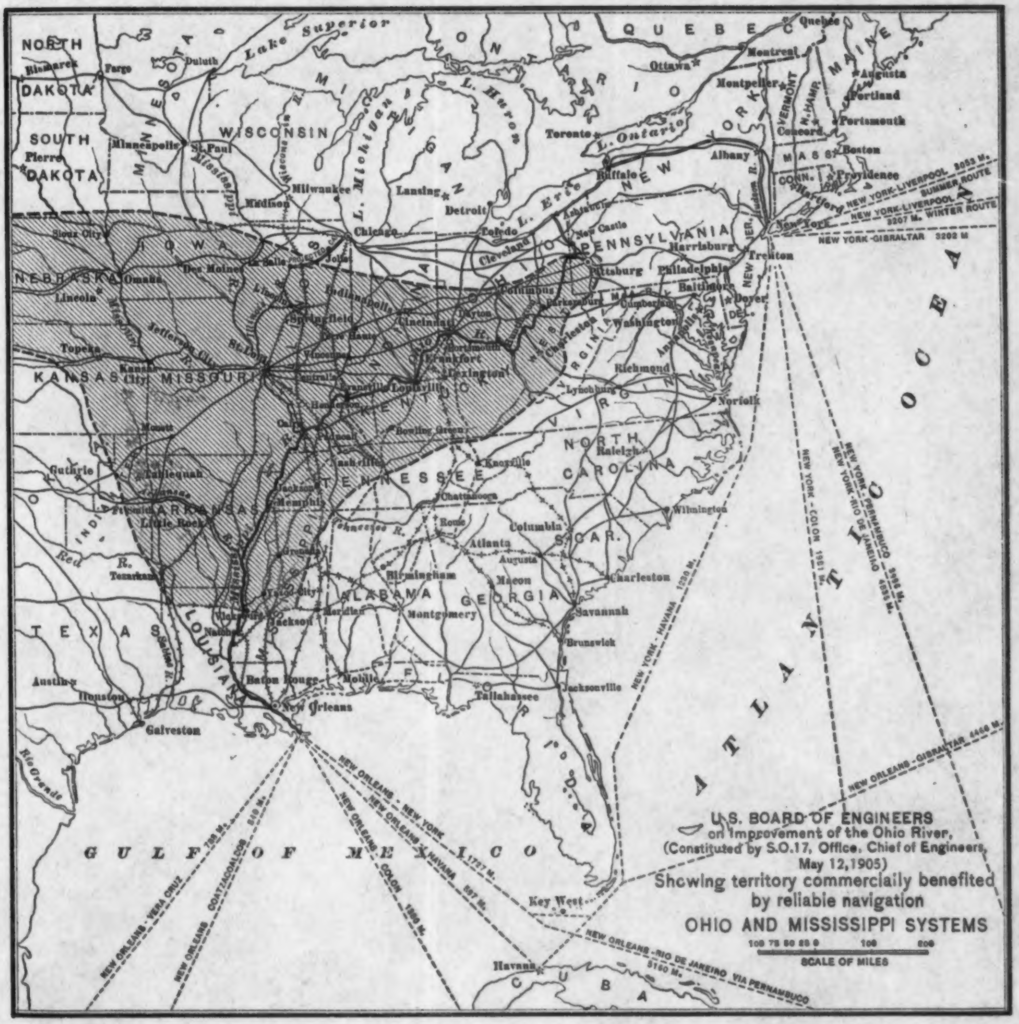
SIBERT ON
IMPROVEMENT OF THE OHIO RIVER.



U.S. BOARD OF ENGINEERS
on Improvement of the Ohio River,
(Constituted by S.O.17, Office, Chief of Engineers,
May 12,1905)
Showing territory commercially benefited
by reliable navigation
OHIO AND MISSISSIPPI SYSTEMS

final destination. How much will be saved by this method of shipment is not stated."

It is thought that the tolls charged on the Panama Canal should be low—not much more than enough to cover the operating and maintenance expenses. The construction cost should be charged largely to National protection, at least until a large traffic is developed.

The map, Plate XXI, accompanying a report of an examination of the Ohio River made by a Board of Engineers, United States Army, and transmitted to Congress in January, 1908, shows the territory that it is thought will be commercially benefited by reliable navigation in the Ohio-Mississippi system.

The Report states:*

"The boundary line of this territory was found by drawing a line through towns to which the freight rate from New Orleans by river and rail at present rates is equal to the present all-rail rates from seaport towns to the same points. Barge freight rate from New Orleans to distributing points such as Vicksburg, Cairo, Evansville, Louisville, Cincinnati, Wheeling, and Pittsburg was assumed at 1.14 mills per ton-mile, which was the rate charged during the year 1905-6. The rail rate used was 4.832 mills per ton-mile from Gulf cities northward and is the present mean ton-mile rate on sugar in carload lots from New Orleans to Chicago, Cincinnati, Louisville, and Pittsburg. The rate used from Atlantic Coast cities inland is that of the Pennsylvania Railroad from New York to Pittsburg, or 7.4 mills per ton-mile, which is the mean of the ton-mile rate on sugar and rice. A rate of 2 mills per ton-mile was used on the Hudson River and through the Erie Canal, and a rate of 1½ mills per ton-mile on the Great Lakes for such commodities as sugar, rice, etc.

"Within the territory shown on the map now lives a population of about 20 000 000 people, 8 504 000 of whom live in that portion of territory lying in the Ohio Valley proper."

Methods of Improvement.

The original method of improvement of the Ohio River included the "removal of snags and rocks; the closing of duplicate channels by low dams; the guiding of the water into a single low-water channel by dikes where the river had an excessive width; and the removal, by dredging, of hard bars and projecting points."

It was only hoped by this method to prolong the navigation season for packet business, enough water not being available for producing

* Executive Doc. 492, 60th Cong., 1st Sess., p. 22.

a depth and width sufficient for towboats and fleets. Economy of transportation on the Ohio being within limits proportional to available depth and width, experience indicated that fleets from Pittsburg to New Orleans should draw at least 9 ft., and that channel widths of at least 300 ft. were necessary at the lowest navigable stage. This navigable condition could only be brought about permanently by materially increasing the low-water flow of the river, or by locks and dams.

The following methods of increasing the low-water discharge have been proposed:

(1).—To obtain the additional water needed during the rainy season by cutting a ditch from Lake Erie to the Ohio River. However, as the river above Marietta, Ohio, was at a higher elevation than Lake Erie, this method was impossible.

(2).—That Lake Chautauqua should be used as a regulating reservoir for the Ohio River. The drainage area of the lake was too small, however, for serious consideration. It was then proposed to use Lake Chautauqua as a reservoir, and to keep it filled by pumping from Lake Erie.

(3).—Colonel Charles Ellet, a civil engineer engaged in work at Wheeling, West Va., made some observations of the discharge of the river there, from which he reached the conclusion that the average annual discharge of the river was sufficient to maintain a continuous 6-ft. navigation, if the water was stored in reservoirs at times of large discharge and permitted to flow at times of small natural discharge. The plan proposed involved the construction of large dams to form reservoirs upon the head-waters. This scheme was reported upon adversely by W. Milnor Roberts, United States Civil Engineer, who was in charge of the Ohio River improvements from 1866 to 1870. Mr. Roberts was peculiarly well qualified to investigate the subject, both because of his efficiency as a civil engineer, and also because of his intimate knowledge of the region drained by the Allegheny and Monongahela Rivers, he having been connected at different times with the Pennsylvania State Canal between Johnstown and Pittsburg; the canal between Erie and Beaver; the Monongahela slack-water system; and surveys for various railroad lines in the regions mentioned.

This plan has recently been revived. It should be remembered, in considering it, that an open-channel depth of at least 11 ft. is now necessary, the trend being always for deeper water. A 14-ft. depth

is being advocated between Chicago and St. Louis. The records seem to indicate that the supply of water is not sufficient to provide such a depth during the entire year in the Upper Ohio River, while there may be sufficient for the Lower Ohio.

The proposition to increase the low-water flow by means of water impounded in reservoirs, had, of course, the double object of thus improving navigation and, at the same time, providing for flood prevention. The extreme irregularity with which floods occur and the navigation necessity of always having full reservoirs at the beginning of the dry season, would result many times in the reservoirs being full when needed for flood prevention. Thus they would not be able to restrain the flood in such manner as to prevent entirely the usual damage.

If flood prevention alone be the function of the reservoirs in the Upper Ohio, the writer believes that they could be made to serve such purpose. During the winter season, when the movable dams were down, the impounded water might be used to prevent the formation of ice in large quantities, by increasing the flow so as gradually to drive out such ice as forms, thus acting as an auxiliary to the present movable dam system. The water of the Chagres River, on the Isthmus of Panama, is to be impounded, in order to control its flow, in a reservoir which will be 165 sq. miles in area. The flood discharge of this river is about one-third that of the Ohio at Pittsburg.

Reservoirs as means of river improvement have always been attractive. Their cost, however, in connection with the danger of their being filled with detritus from the surrounding hills, has prevented their adoption generally as a system of river improvement, both in the United States and in Europe, and that on streams where the slope was such that commerce could obtain the full benefit, if the necessary depth existed.

A company was organized in 1855 for the purpose of experimenting on the Ohio River with a system of improvement which had been devised by General Herman Haupt. The Company proposed to prosecute the work at its own expense and on condition that no payment should be made by the Government unless the plan proved to be a success. In brief, the plan of improvement involved the use of low dams and open chutes, about 300 ft. wide, inclosed on one side by the natural bank of the river and on the other by a longitudinal bank or

mound. This plan was reported upon adversely in 1870 by Mr. Roberts, and again in 1880 by a Board of Engineers.

Mr. Alonzo Livermore proposed that low dams be used, but that, instead of locks, long chutes should be provided. These chutes would contain division walls and enlargements located so as to decrease the velocity and the discharge of water through the chutes.

Mr. Roberts investigated all the above-mentioned plans and then decided to recommend that the river should be canalized, locks and fixed dams being used, each dam being provided with a navigable chute for use during freshets of moderate height.

A few years later, the late W. E. Merrill, M. Am. Soc. C. E., Major, Corps of Engineers, United States Army, investigated the subject of a radical improvement of the river and agreed with Mr. Roberts that a lock and dam system was the proper solution of the problem. He stated that the advantages of a canalization scheme over any other proposed method of improvement were as follows:

(1).—It has been long tried and is now in use on the Monongahela River, where it meets the demands of the same commerce that navigates the Ohio.

(2).—There are no great hazards connected with the system, since the dams are low and the destruction of one would not necessarily injure the one next below.

(3).—It is known positively that locks and dams can be built that will answer the purpose fully, and their cost can be determined beforehand with very fair accuracy.

(4).—There would be no damages from overflow, or destruction of property of any kind.

(5).—No special care is needed in the use of the slack-water system. The pools are themselves reservoirs containing the minimum quantity of water needed, and at the exact place where it is to be used.

(6).—The cost of the system would probably be less than that of the other (reservoir) system.

(7).—The pools would make excellent harbors for all river craft, an improvement that is greatly needed at the large cities, and especially at Pittsburg.

There was one great objection, however, to the construction of any system of dams such as had ever previously been built in the United States. At times of freshets, all the coal shipments were made in fleets

which were too large to go through a lock of any reasonable dimensions, without being divided into two or more lockages. When the fleets are made up, the barges are fastened together very securely by cables in every direction, and the delays which would be caused by making and unmaking a fleet at each lock would be very great. (Plate XX.) The coal shippers were vigorously opposed to the construction of any system of dams which would not permit the passage of fleets down stream, at times of freshets, without being broken up and reformed at each dam. It was believed that the influence of these interests was such that they would be able to prevent the appropriation by Congress of funds for any system of dams which would change the established system of coal carriage during high stages of water.

A Board, consisting of Majors Weitzel and Merrill, then made a careful study of the whole subject. A system of fixed dams was studied first, each dam to have an opening, 4 ft. deep and 200 ft. wide, cut in the top, with an inclined plane below the dam, and built so that, at high stages, coal fleets could be passed through the opening and down the inclined plane into the lower pool. The opening in the dam was to be closed by some form of movable gate which could be operated easily and quickly.

The Board studied the French and other systems of movable dams, but considered that there might be objections to the use of Chanoine wickets on the Upper Ohio, which did not exist in the case of the rivers of France. It was believed that the large quantity of drift and ice found in the Upper Ohio might be very injurious to the parts of a movable dam of the French type. Major Merrill's plan was finally adopted in part. This plan provided for 13 locks and dams between Pittsburg and Wheeling, built for a minimum depth of 6 ft., the dams to be of the movable type, with navigable passes 400 ft. wide, to be closed by Chanoine wickets, and high and low weirs, to be closed with either Desfontaines' wickets or Brunot's gates; the locks to be 78 ft. wide by about 630 ft. long, with lifts of from 4 to 7 ft. The first dam was located at Davis Island, five miles below Pittsburg. The plans for the first lock and dam were subsequently modified so that the lock as built was 110 ft. wide and 600 ft. long, and the weirs were closed by Chanoine wickets of shorter lengths than those of the pass. All wickets were originally planned to be maneuvered from service bridges. Davis Island Dam was commenced in 1878, and opened to navigation in 1885, and its cost was about $910 000.

The operation of the dam at Davis Island having been successful, in 1888, Major Merrill proposed the immediate construction of two more dams of the series. The Board of Engineers went still further and recommended the extension of the slack-water system as far as the mouth of the Beaver River, by the construction of four of the dams previously mentioned in Major Merrill's project of 1874. The first appropriation for building Dam No. 6 was made in 1890 and the last in 1902, and the lock and dam were completed and put in commission in the summer of 1904.

The first appropriation for the construction of Locks and Dams Nos. 2, 3, 4, and 5, was made by Congress in June, 1896, and the last in 1905. These latter dams were completed in 1906 and 1907. The time taken by Congress to appropriate the money for these dams is specially mentioned, because engineers are often blamed for the slow building of structures for which legislative bodies have not provided the funds, for which latter action engineers are in no way responsible.

In the construction of such a system of improvement as described in this paper, the building of each lock and dam complete should be provided for by one contract. This would prevent the delays due to entering into many contracts at different periods of the construction, and those due to collecting and erecting different sets of contractor's plant, etc.

Each lock and dam complete should be built in from 3 to 5 years, and there is no limit, of an engineering nature, to the number of locks and dams that can be contracted for at the same time. Of course, this means that all the funds necessary to complete a lock and dam must be authorized prior to the commencement of the work.

The River and Harbor Bill, passed in the spring of 1905, authorized changes to be made in Locks and Dams Nos. 1 to 6, so as to provide for a navigable depth of 9 ft., instead of 6 ft., as originally planned.

The harbor of Pittsburg had been dredged so as to permit navigation by boats drawing between 9 and 10 ft., and, therefore, it was only necessary to change the plans of the dams which had not been completed, viz., Nos. 2, 3, 4, and 5. The changes made comprised the raising of these four dams by means of longer wickets and higher bear-trap gates, and the lowering of the lower lock sill at Dam No. 5. No changes had to be made in the completed dams, Nos. 1 and 6, but,

in order to permit of the extension of the harbor of Pittsburg to include a good natural deep-water pool a short distance below Dam No. 6, the lower lock sill at that dam was lowered. This will allow coal boats to be taken from Pittsburg during the navigable season, at any stage of the water, down as far as this good natural harbor, there to await a coal-boat freshet.

The Act of Congress, approved March 3d, 1905, made provisions for a survey of the Ohio River from Pittsburg to Cairo, with a view to the radical improvement of the river throughout its entire length. The report of the Board of Officers appointed to make the survey has been submitted to Congress. This report recommends the canalization of the river from Pittsburg to Cairo, so as to provide a navigable depth of 9 ft.; the locks and dams are to be built of practically the same type as those already constructed in the upper portion of the river; the Louisville and Portland Canal is to be widened, and a duplicate lock is to be placed therein. The estimated cost of the entire project was \$63 731 488, in addition to the appropriation already made, viz., \$9 281 376.

The locks recommended are to be 110 ft. wide and 600 ft. long in the clear. The dam is to consist of a navigable pass from 600 to 700 ft. wide, an automatic part, probably of the bear-trap type, and a series of movable weirs of the Chanoine, or other well-tried type; the proportion of weir and automatic part is to be determined by the conditions existing at each dam.

Thus far the order of building has been first to construct locks and dams below the important cities and tributaries, so as to provide good harbors as soon as possible with the funds available, and thereby enable river commerce to connect with existing railroads. Thus, Davis Island Dam was built below Pittsburg; and, at present, Dam No. 13, below Wheeling, No. 18, below Marietta, and No. 37, below Cincinnati, are under construction.

The following rules were promulgated by W. H. Bixby, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. A., for the location of locks and dams in the upper river:

"1st. To allow room in the river for a navigable pass at least 600 feet wide, with its centre nearly in line with the existing channel at tow-boat stages of water.

"2nd. To allow room in the river for weirs of at least 240 feet total length, so placed as to be as much as possible protected from injury by floating drift.

"3rd. To allow room in the river for passage of coal tows around each half of the dam during construction of the other half.

"4th. To allow of the sill or bottom of the navigable pass being placed at least slightly above the present bed of the river, and yet at least as far below the water surface as are the channel bars, which at coal-boat stages determine the available depth of the river in the pools above and below.

"5th. To avoid positions where the coal-tows must be flanked or placed obliquely to the channel line by reason of river bends, or oblique currents.

"6th. To allow of the view of dams by approaching boats for distances of at least half a mile both up and down stream.

"7th. To avoid positions near shifting sand or gravel bars, and mouths of sand-bearing rivers, creeks, and runs, unless such bars and streams be below the dams on the weir side.

"8th. To allow of positions below the mouths of large tributaries like the Little Kanawha, Great Kanawha, Guyandotte, Big Sandy, Scioto, Licking, and Big Miami, near enough to extend navigation properly to the nearest dam in such tributary, but yet far enough below the mouth of such tributary to avoid trouble from deposits and flood overflows, and to allow the proper handling of boats entering or leaving such tributaries.

"9th. To avoid cutting up or otherwise injuring the existing harbors of large cities, especially Parkersburg, Pomeroy, Point Pleasant, Gallipolis, Huntington, Catlettsburg, Ashland, Ironton, Portsmouth, Cincinnati, Covington, and Newport.

"The first, second and third conditions prevent locations in narrow parts of the river, and especially at islands where the bank channel is bare at low water, and of insufficient depth at coal-boat stages. The third condition prevents location within a half mile of the ends of islands, or high mid-river bars. The fifth and sixth conditions prevent locations at sharp bends.

"While it is not to be expected that all of these conditions can be fulfilled at each location, still past experience on this river, combined with the information so far secured, indicates that these conditions can, in general, be complied with, and that departures from the same will be the exception rather than the rule."

It is important, in planning any system of slack-water navigation, to foresee and provide for any increased depth which may be demanded in the near future. It is better to design the locks so as to accommodate a greater draft than that indicated at the time of construction, in order that increased depth may be easily obtained in the future by dredging in the pools. The extra cost of a lock so built is limited to the excess cost of the lower gate over the decreased cost of the lower

sill foundations. For this reason, any system of locks and dams should be designed so as to provide the draft needed, without the use of auxiliary dredging, leaving the deepening of the pools to be done later, to provide for a possible demand for increased depth.

It will be remembered that the dams originally proposed for the Ohio were fixed dams provided with a navigable chute. The requirements of the coal-fleet navigation made fixed dams which were not provided with some kind of navigable pass out of the question.

A movable dam to be used on the Ohio River should conform to the following:

It should be easily operated and free from complicated mechanism.

It should be capable of being lowered rapidly, without chance of failure.

Its parts should be so strong and simple as to prevent damage by ice and drift. This consideration is of very great importance on the Ohio, where the severity of the winters must be kept in mind in selecting a type of movable dam.

It should be capable of being easily operated, so as to pass small freshets without the necessity of lowering the entire dam and without lowering the pool below normal level.

It should afford a navigable channel at least 600 ft. wide, with the dam down, and the passage should then have a depth of water equal to that on the bars above and below the dam. It is thought that the combination of Chanoine pass and weirs, with bear-trap automatic portion, fills these conditions better than any other.

The lift of the dams planned, up to the present, has been limited to about 8 ft. as a maximum. This was originally based on the experience in France. With these small lifts and a navigable stage of only 6 ft., the wickets for the navigable pass were of a length not greater than 14 ft. The increase in navigable depth on the Ohio, from 6 to 9 ft., caused the use of wickets as long as 18 ft. These have been maneuvered satisfactorily, and it is possible that Chanoine dams of a 10-ft. lift could be satisfactorily operated.

A wicket 16 ft. long was adopted in the project last recommended to Congress. If the construction engineers can safely lengthen these wickets, a smaller number of dams will be needed.

There are no lift walls in the locks, both gate-sills being at approximately the same elevation as the sill of the pass. This is advan-

tageous, because it permits boats to go through the lock at times when the dam is down and there is not much water in the river. When an upward-bound packet meets a tow going down stream through the pass, it is often desirable for the packet to pass up stream through the lock.

There are no guard walls prolonging the river wall of locks, because it has not been believed that they are necessary.

Where the foundations of the river walls of the lock are such that no serious scour may be anticipated on the outside, due to the operation of the dam, it is customary to let the dam start from about the center of the lock wall, thus affording ample room for emptying and filling the valves, which are placed in the outside wall itself.

Whenever there is danger of scour at the place designated above, it is customary to let the dams abut against the wall as near the lower end of the locks as possible, and, at the same time, to provide the necessary filling and emptying arrangements. Eddy currents below movable dams should not be very serious, since the dams are down at all higher stages of the river.

Before a sudden rise, it may be essential to commence lowering the navigable pass at the end next to the lock, although this should not be done unless absolutely necessary. If it is done, the scouring effect of the current alongside the lower end of the river wall is very great. It should not be concluded that, because movable dams are down during the higher stages, scour is not probable.

In case of accident to the lock gates, the rush of water through the lock would probably undermine the walls, unless they were properly protected against scour. Moreover, during construction, there are times when the cross-section of the river at the site of the dam is very much reduced by coffer-dams, etc., and the currents through the openings between obstructions are such as to produce very dangerous scour. The wall foundations should be protected against any of these contingencies.

At Lock No. 4, during construction, the entire width of the river was at one time closed, except about 430 ft. which was divided into two channels. One of these channels, next to the lock wall, was only 220 ft. wide. The scour along the lower end of the river wall, caused by the severe currents through this narrow passageway, threatened to undermine the lower end of the wall, and the scour was only stopped by the prompt placing of a large quantity of rip-rap, thrown by hand from the top of the wall.

The principle which should govern the height of the walls above the pools is that they should be of such a height as will permit a boat to lie alongside them with its guard below the top of the wall. This would indicate that 5 ft. above the water is the proper height to accommodate packet-boats, the guards of which are generally from 4 to 4½ ft. above the water surface, but, of course, this depends on the load carried. Towboats have much lower guards. As the lock may be used until the upper pool rises to a height of about 1 ft. above the crest of the dam, it would be better to build the upper guide wall and the land wall of the lock to an elevation of about 6 ft. above the level of the normal upper pool. The river wall need not be built so high, because it is unusual for packets to lie alongside that wall while making a lockage, and, if necessary, their fenders can be used at any time. In fact, the dams in the stretch of river near Wheeling have river walls of less elevation that the land walls; but, as originally built, the walls of the locks in the upper part of the river were uniformly 5 ft. above normal pool level. The increase in pool elevations, caused by the change from a 6- to a 9-ft. navigable depth, decreased the height of the walls above the pools and necessitated raising some of the walls. In most cases, the land wall and lower guide wall were raised, the river walls not being raised. The lower guide walls were originally built to an elevation of 5 ft. above normal lower pools, which provided the same clearance as the other walls, when the pools were at normal level. However, this did not take into consideration that when there is considerable flow over the dam, the lower pool rises almost twice as fast as the upper pool. It would seem then that, for a symmetrical design, the lower guide walls should be built to an elevation of about 7 ft. above normal lower pool, if the land wall is to be 6 ft. above normal upper pool. A guide wall should be at least as long as a tow that can pass through the lock in one lockage. In the latest dams on this river, the guide walls are from 600 to 700 ft. long.

Lock-Gates.

The Ohio River locks are differentiated from most other locks in the United States principally by the type of lock-gate used. The gates are of the rolling type designed by the late Major W. E. Merrill. When out of use, a gate of this type is housed in a recess in the bank; when needed, it is run across the lock, thus closing it, the screen serv-

ing as a gate. As originally planned the locks were to be 78 ft. wide. It was not believed to be practicable to make them wider, because of the small height and the great length of the miter-gates. However, when it was deemed necessary, later, to increase the width to 110 ft., a form of gate suited to such a wide span and small height was designed. These rolling gates run on tracks in much the same way as an ordinary railway car. The principal parts of a gate of this type are:

First.—The top truss which serves to carry a portion of the water pressure to the lock walls.

Second.—The vertical water-screen which serves to close the lock and transfers one-third of the pressure to the top truss and two-thirds to the track.

Third.—The trucks, wheels, etc.

The original gates at the Davis Island Lock were made of pine timber. The top truss was of the Howe type, supported on posts so as to lie above the normal level of the upper pool. The posts extended below the level of the track, and their lower ends served as flanges for the wheels, which were of the plain-tread type, traveling on a flat-rail track, the gauge of which was 11 ft. 6 in. The spacing of the rows of posts was such as to provide for a 2-ft. lateral movement of the gate; for the gate to move laterally, however, it was necessary for it to slide on the wheels parallel to the axis of the lock. This sliding created great strains on the lower part of the gate, and resulted in the breaking of many wheels and axles. The gate was moved back and forth by two chains, each chain being fastened at one end to an end of the gate and wound around a chain drum at the entrance to the recess. The chains were rigidly attached to the gate, and this mode of fastening was found to give much trouble. If the gate struck a submerged snag or rock on the track, it stopped moving instantly, and something broke before the engine could be stopped. Wheel axles were broken several times, and, on one occasion, when the gate could not be moved out of the recess, it was found that the outer axle was broken, the lower wheel of the second axle off, and the axle out of its bearings.

After the original wooden gates had served for 12 years they were replaced by steel gates. In these new gates the top truss is of the Pratt type, the wheels are flanged, and a lateral movement of the gate is provided for by hanging the framework upon each axle by eye-bar hangers, which allow the frame to swing laterally like a pendulum. This ar-

PLATE XXII.
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FIG. 1.—LOCK NO. 6, OHIO RIVER.

FIG. 2.—GATE RECESS, LOCK NO. 6, OHIO RIVER.

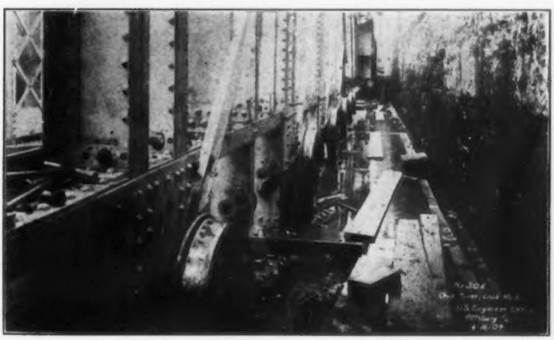

FIG. 3.—GATE FOR LOCK NO. 6, OHIO RIVER.

rangement prevents the friction of the bottom bearing from interfering with the opening of the gate. When the pressure is on the gate, it swings down stream about 2 in. until the bottom-bearing piece rests against the up-stream side of the down-stream rail. When the pressure is released, the gate swings back so that the bearing strip is about 2 in. away from the rail. In the new gate, rectangular valves are used instead of the circular butterfly valves of the original gates.

The latest gates are those at Locks Nos. 2, 3, 4, 5, and 6. They are of steel, and are somewhat similar to the steel gates at the Davis Island Lock. The top truss is also of the Pratt type. The water-screen consists of horizontal white oak planks bolted to vertical, 15-in., 42-lb. I-beams, which rest, at the top and at the bottom, against horizontal members riveted at each end to the down-stream posts of the gate. Below the intermediate horizontal member and the bottom water-coal strip, are the gate valves. There are two of these valves in each panel, making eighteen in each gate. The valves are rectangular, of the horizontal-axis, butterfly type, approximately 3 by 4 ft. in size. They are made of structural steel closing against oak cushions, and are operated by hand from the top of the gate by racks and pinions. The valves were not constructed to be operated by power, because the locks were provided with filling and emptying valves in the river wall, and the gate-valves are not to be used except in case of accident to the wall-valves, and possibly to assist in flushing deposit out of the lock chamber and the tail-bay. (Plate XXII.)

The pendulum arrangement for the lateral swing of the gate is much the same as in the later Davis Island gate already described. The connection between the superstructure of the gate and the trucks is through standard car springs which serve to prevent shocks to the gate when the wheels run over obstructions on the track. The wheels are standard car wheels, slightly modified in the tread so as to fit the level gate-tracks. The water-seal at the bottom is formed by a wooden bearing strip. The water-seal at each end of the gate is formed by a pivoted pipe arranged so as to be swung by the pressure of the water until it closes the opening between gate and wall. The operating chains are 1¼-in. crane chains fastened at each end of the gate to a tug-lever, which is arranged with spring control so as to diminish the shock due to starting and stopping the gate. In order to prevent breakage of the

chain, the connections to the tug-levers are made by shackles and pins of such a size that the pins will break before any link in the chain can possibly fail. The gate is moved back into the recess and out across the lock by the rotation of the chain drum at the entrance to the recess. This drum and its shaft are rotated through a system of gearing by a 10 by 12-in. engine of the horizontal, double-cylinder, reversible type, operated by compressed air, at Locks Nos. 2 to 6. At Davis Island a 12 by 16-in. steam-driven engine is used.

In the design of these gates, no estimate was made of the stresses which would result in case a gate should be struck by a boat entering the lock. All truss and screen members were designed to support the static water load, under the assumption that the lower pool had been drawn down to sill level. Safe working unit stress for steel members was assumed at 10 000 lb. per sq. in.

The collection of drift in the gate recesses and in the gates, has given great trouble at Davis Island. This is due largely to the fact that that lock is filled partly by valves in the upper recess, and is emptied partly by valves in the lower recess. The operation of these valves creates currents toward the recesses, which draw into them the drift accumulated in the forebay and in the lock-chamber. It is not believed that as much difficulty will be encountered at the other locks, as they are filled and emptied entirely through valves in the river wall. However, in order to prevent any drift from entering the recesses, it has been proposed that the gates built hereafter be provided with pervious screens on their up-stream sides, placed so that when the gate is moved, these screens will pass close to similar screens projecting from the up-stream sides of the recesses at the entrances. All drift can then be locked through, and none will be allowed to enter the recesses.

The gates at Locks Nos. 2 to 5 can be opened or closed in from 1½ to 2 min. The average time of operation at Lock No. 6, which has been in use for three seasons, is 2 min. The gates at Lock No. 6 have been moved while under a 6-in. head, the lower gate being closed while the filling valves were open and while there was considerable current in the lock-chamber.

The recesses in which the gates rest when the dam is down, and from which they emerge when the lock is put into operation, are 125 ft. long and 20 ft. wide in the newest locks under consideration. The recesses of the lock at Davis Island are so narrow that there is only

about 1 ft. clearance on each side of the gate. This has often proved to be too small to permit of making the repairs to the gates, which are constantly required. On one occasion it was necessary to raise the gate 2 or 3 ft. off the track in order to make repairs. When a gate is erected in place on the tracks, which is the usual method, considerable room is needed during the erection. Moreover, if the gate be erected on the track in the recess after the lock-chamber has been flooded, a sump should be provided inside the entrance to the recess, and there should be room for the necessary suction pipes, etc. It is often desirable to use a horizontal, centrifugal pump in unwatering the recess, and if such a pump is not placed on some kind of floating support, it is desirable that it be placed in the recess. It is well, therefore, to make the recess large enough to permit this arrangement. It is believed that the recesses should be provided with vertical slots in the side-walls near the entrance so that horizontal timbers can be placed in the slots, thus forming the sides of a coffer-dam.

The gate-recesses at Locks Nos. 2 to 6 are wider, but not longer, than those at Davis Island. It is believed that a recess should be about 135 ft. long for a lock 110 ft. wide, the gate being about 118 ft. long. It has been noted that the gauge of the gate tracks is 11 ft. 6 in. They were so constructed, but, just prior to the erection of the gates, at Locks Nos. 2, 4, and 5, it was found that the gauge of the track at the entrance of the recesses was from 1 to 2 in. too small. Inasmuch as no greater clearance than $\frac{3}{8}$ in. had been provided in the gauge of the gate tracks, it was necessary to tear up and reset a portion of the track. It is believed that expansion of the concrete in the long land walls and guide walls was responsible for the narrowing of the track. To prevent any movement of the track rails, it has been proposed to use ties made of two channels, back to back, with the rails riveted to an interior gusset plate at each end of the tie, these steel ties to be used instead of the former wooden ties.

There having been considerable trouble at the Davis Island Lock, caused by the collection of drift in the gate recesses, drift chutes were built when the guide walls were rebuilt of concrete. In Locks Nos. 2 to 6, the drift chute is a tunnel running from the rear of the upper gate recess down past the rear of the lower gate recess (with which it is connected) and emptying into the tail-bay of the lock. The entrances to the drift chutes are provided with movable doors, and are intended for

flushing the drift out of the recesses. The construction of the drift chutes is very expensive, especially so when rock is found at a high elevation. It seems that it might be well to omit them and solve the drift problem by collecting the drift below the mouths of the tributaries, and not be bothered with the same drift at every dam on the river (Fig. 1). Of course, the greatest quantity of drift is running at times of freshets, when the dams are down, and the only drift to be cared for by the method proposed would be that which is floating on the pools. This will probably decrease in quantity.

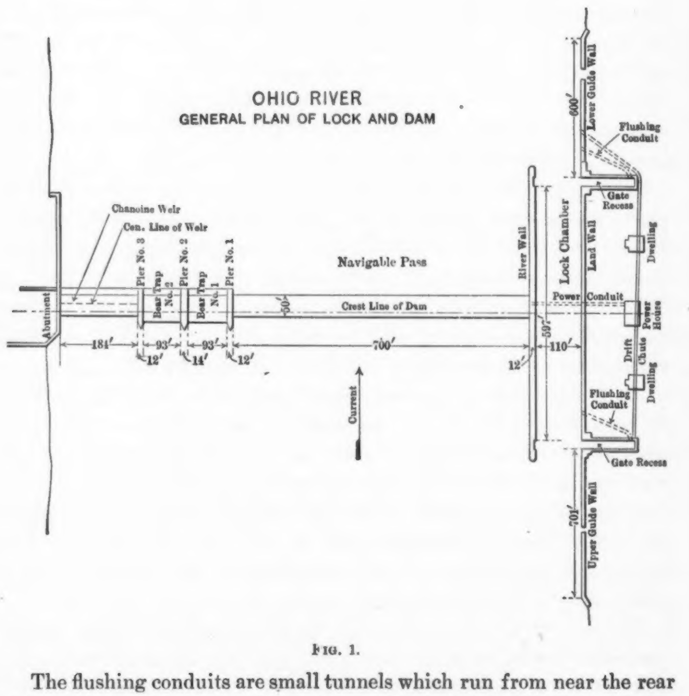

FIG. 1.

The flushing conduits are small tunnels which run from near the rear of the gate recesses and empty into the lock-chamber and tail-bay. They are intended to be used in flushing all sedimentary deposit out of the recesses. They are provided with Stoney gate-valves at the entrances, and, if used when the corresponding gate is across the lock-chamber,

a head equal to the lift of the dam is available for flushing purposes. They will work very nicely if the deposit is so small in quantity as to permit the gate to be run across the lock-chamber, but if the gate can be run across the lock-chamber, why is any flushing necessary? It is noteworthy that, although flushing conduits were built at Lock No. 6, they have never been used, and the lock has been operated continuously for three seasons. After the high flood of March, 1907, the quantity of deposit in the gate recesses was very great. In some of them there was found a sedimentary deposit 7 ft. deep, and, from the two gate recesses at Lock No. 2, approximately 1 400 cu. yd. had to be removed. The deposit at all the locks after this flood was so great that, before any gate could be moved, it was necessary to coffer off the recess and remove the deposit. The flushing conduits were of no use because no head was available, the dams being down and the gates stuck fast. In order to ensure the satisfactory working of the lock-gates and to increase their durability, it is believed that it is wise to pump out the gate recesses every spring prior to the first raising of the dams. The débris can then be removed, the gates repaired, if necessary, and painted.

Gate recesses are provided with roofs built either of reinforced concrete or of I-beams supporting ½-in. steel plates. The last-mentioned type is believed to be the better, as it can be easily removed prior to making any needed repairs to the gates. The roofs are placed at such a height as will permit a man to walk on top of the gate and push the drift out of the recess, or into the drift chute.

Each lock is provided with a conduit which runs across under the lock from the bottom of a well in the power-house to the bottom of another well in the river wall. It is in this conduit that the pipes are laid which supply pressure for working the river-wall valves and the bear-trap gates. In placing or repairing these pipes, it is necessary to pump out the conduits, and they should be provided with a sump at the bottom of one of the wells. If used at all, these power conduits should be placed so low that they cannot be injured by any dredging that may be required in cleaning out the lock-chamber. In the later designs, the conduits have been omitted, the power pipes being placed in troughs in the gate-track foundations.

At Davis Island, the lock is filled through seven, circular, butterfly valves, 4½ in. in diameter, in the river wall, and the same number

in the down-stream wall of the upper gate recess. The lock is emptied by seven valves of the same kind and size in the down-stream wall of the lower gate recess, which all discharge into a large conduit emptying into the tail-bay. Gate-valves were also used. The wall-valves were provided with vertical axes, and were turned by lever arms moved by a large hydraulic jack, one jack turning seven valves. The valve stems had rigid connections with the hydraulic jacks. Because of obstructions getting in the valves, most of the valve stems became twisted and the valves would not close, thus making the leakage great. A spring compensating device was originated, which permitted any one valve to remain open while the others were closed. At Lock No. 6, considerable trouble was had with the breaking of springs in this compensating device, and individual jacks, arranged in a parallel closed circuit, were adopted for use at Locks Nos. 2, 3, 4, and 5, each valve being provided with one of these little engines. The jack consists of a cylinder of Shelby drawn-steel tubing, 5½ in. in diameter, provided with cylinder heads, cross-head guides, etc. Motion is given to a lever arm by a connecting rod operated by a piston of 30-in. stroke. The parts were proportioned so that the valve shafts could not be twisted by any force which could be transmitted to the lever arm by the pistons. The shaft of any valve not operating would not be injured, therefore, while the others turned. The valve shafts are vertical, and made of cold-rolled steel, 3¼ in. in diameter.

The valves at all the locks, except those at Davis Island, were placed in the river wall of the lock—sixteen above the dam, for filling the lock-chamber, and sixteen below the dam, for emptying it. The valves are circular, butterfly valves, 4½ in. in diameter, of cast iron, in a single piece, with horizontal stiffening ribs.

The individual jacks have been highly successful. They are operated by water, except in the coldest weather, when oil is used. The pressure is obtained from a duplex pump in the power-house. The piping is installed so that, while water under pressure is supplied at one end of the jacks from the common supply line operating the pistons and valves, the water on the other side of the pistons flows into the other supply line and back to the tank in the power-house. A four-way valve controls the flow, so that the direction of motion may be reversed.

FIG. 1.—MANEUVERING BOAT AT LOCK NO. 2, OHIO RIVER.

FIG. 2.—OPENING THE DAM AT LOCK NO. 2, OHIO RIVER.

Power-Plants.

At Davis Island, the steam for operating the lock-gate engines was generated in a boiler at the lower end of the locks; the steam for the upper gate engine was piped about 600 ft. to the engine; so much trouble was had from condensation that a boiler was installed at the upper recess. At Lock No. 6 the boilers were assembled in the power-house, which was opposite the middle of the lock, and steam was piped a distance of about 300 ft. to each gate engine; here again considerable trouble was encountered on account of condensation. A single-stage air compressor, with a capacity of 200 cu. ft. of free air per min., had been installed to assist in moving the bear-trap gates. This was replaced by a two-stage steam-driven compressor, with a capacity of 400 ft. per min., and larger storage tanks were installed. Thereafter the gate engines were operated more satisfactorily with compressed air.

The valve engines at Davis Island were originally operated by water pressure, supplied by elevated tanks under a head of about 62 ft. It proved to be hard to move the valves when under a head exceeding 5 ft., and later the pressure was obtained directly from steam pumps.

The plants at each of the recently completed locks consist of two Westinghouse, three-cylinder, vertical, gas engines, operating on natural gas, and two chain-driven, Blaisdell, two-stage air compressors, each having a free-air capacity of 500 cu. ft. per min., at a pressure of 100 lb. The operation of the bear-trap is facilitated by air forced into the lower leaves, direct from the air tanks. The gate engines are also operated by compressed air. The filling and emptying valves are operated by liquid pressure, as already stated, the pump being operated by compressed air. For maneuvering the gates, it is simply necessary to have a man at the throttle of the gate engines, but all operation of valves is from the power-house. Cooling water for the engines and compressors is obtained from elevated tanks supplied by an air-driven pump in the basement of the power-house.

One great advantage in operating the valve engines by liquid pressure is that, in case of accident to the supply of gas, a small boiler can be coupled quickly to the supply pipe of the operating pump, and the valves can be operated in the same way as when the pressure pump was air-driven. If the valve engines were operated directly by air pressure, this could not be done.

The Dam.

The dams are principally of the Chanoine wicket type. The part of the dam over which navigation passes when the dam is down is called the navigable pass. In these dams, the navigable pass is located next to the lock, so that entrance to the lock will not be disturbed by currents from the weirs, etc. Each dam contains regulating weirs of the bear-trap variety. They are separated from the abutment shore by a weir of the Chanoine type, except at Lock No. 6, where there is a weir of the A-frame type. The current through the bear-traps is so swift and the scour below them so great, that, if they are placed next to the bank, considerable extra expense must be incurred for bank protection.

In raising one of these dams, the pass is generally closed, first by the maneuvering boat; then the Chanoine weir is closed by a winch running on a service bridge, and the bear-trap, or automatic part of the dam, is closed last—the raising of the rest of the dam having created head enough to permit the raising of the bear-trap gates either by hydrostatic pressure alone, or assisted by air. (Plate XXIII.)

The requisite weir area is important. It should be sufficient to permit of closing the pass without causing a rise of water surface which will prevent the maneuvering boat from raising the last wickets, the difficulty being in catching hold of the wicket.

While the pass is being raised, discharge is taking place through the Chanoine weir, through the bear-traps, and through the part of the pass not yet raised, as well as through the spaces between the wickets already raised. In calculations, all these points must be considered. Considering now the lowering of the dam because of a rise in the river, the discharge area should be manipulated so as to pass the stream flow without the necessity of lowering all the dams until the discharge becomes large enough to maintain a natural stage of 9 ft. in the river. The upper pool may be allowed to rise until it is about 1 ft. above the crest of the dam, without preventing the lowering of the dam from a maneuvering boat.

It is important that the amount of automatic weir area be not made so great that the flow of the river through the openings can take place without creating head enough to operate the weirs.

In designing the foundations of the dam, the height of the sills or highest fixed parts must be first decided. This is governed by the desirability of not interfering with the normal flow of the river and

PLATE XXIV.
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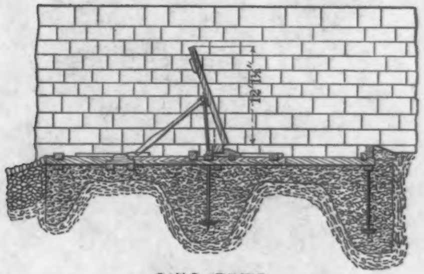
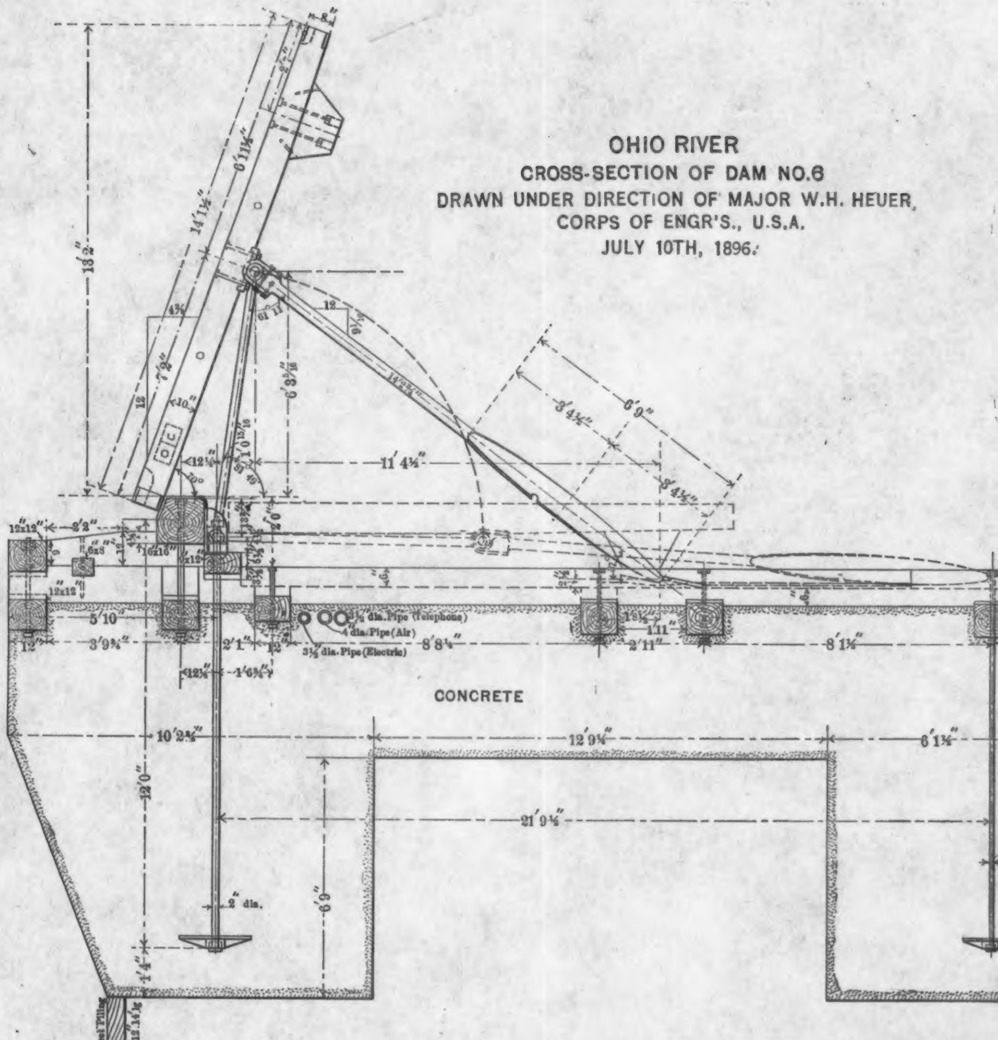
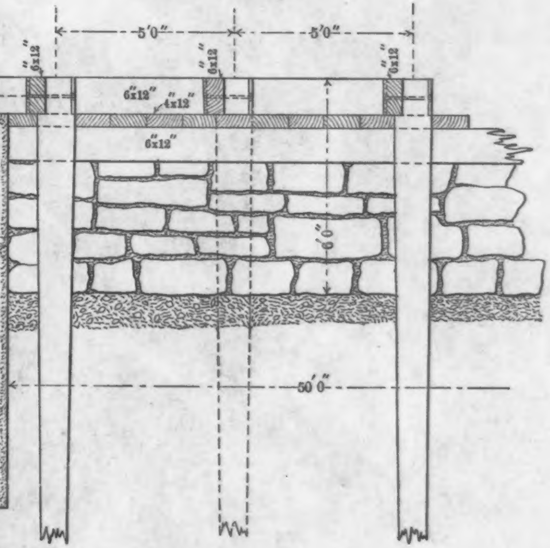


OHIO RIVER
CROSS-SECTION OF FOUNDATION OF DAVIS ISLAND DAM (DAM NO.1)
DRAWN UNDER DIRECTION OF COL. W.E. MERRILL, CORPS OF ENGR'S., U.S.A.
DATE NOT KNOWN

OHIO RIVER
CROSS-SECTION OF DAM NO.6
DRAWN UNDER DIRECTION OF MAJOR W.H. HEUER,
CORPS OF ENGR'S., U.S.A.
JULY 10TH, 1896.





also by the requirements of navigation. The sill of the pass should be placed at least as far below low water as the crests of the nearest bars above and below the dam. If the sill is placed too low, there will be unnecessary trouble in handling the longer wickets, and the foundation will cost more because of being further below the surface of the water. If the sill of the pass is placed too high, it may interfere with open-river navigation.

The sills of the weirs are placed at higher elevations than that of the pass. A Chanoine weir provided with a high sill will have short wickets, and can be easily operated so as to assist in passing small rises without lowering the entire dam, and thus lowering the pool.

The various types of foundations used in these dams are shown on Plates XXIV, XXV, and Figs. 2 and 3.

The general considerations which should govern in designing a dam foundation are, that there should be a water-tight surface at the up-stream face, and that the remainder of the dam should be constructed so as to maintain the water-seal in place and prevent its disturbance by scour below the dam or underneath it. In all these dams the water-seal is practically the same. The dams differ in the style of support for the concrete, and in the character of the protection against scour below the dam. Below Dam No. 2, and below portions of Dams Nos. 3, 4, and 5, stone-filled apron cribs were used, heavy rip-rap being placed down stream from the cribs. Below Dam No. 6, and below part of Dam No. 4, protection against scour is afforded by a wide apron of rip-rap held in place by piles driven in quincunx order. This style is similar to that used in the dams on the River Yonne. Below a portion of Dam No. 5, a wide concrete apron, in which were embedded old wire ropes, was used.

The question of scour below dams is of great importance. It has been necessary at Davis Island on many occasions to place rip-rap below the dam, at one time whole barges loaded with stone being sunk as a protection. The scour is especially bad both above and below the bear-trap gates. Soon after Dam No. 6 was put into operation, scour developed to such an extent around the piers and bear-traps that it reached to the bottom of the concrete at the head of the piers, and just below the protection apron, the scour went to bed-rock, about 31 ft. below low water. It was necessary to place about 2 000 tons of rip-rap to stop this scour.

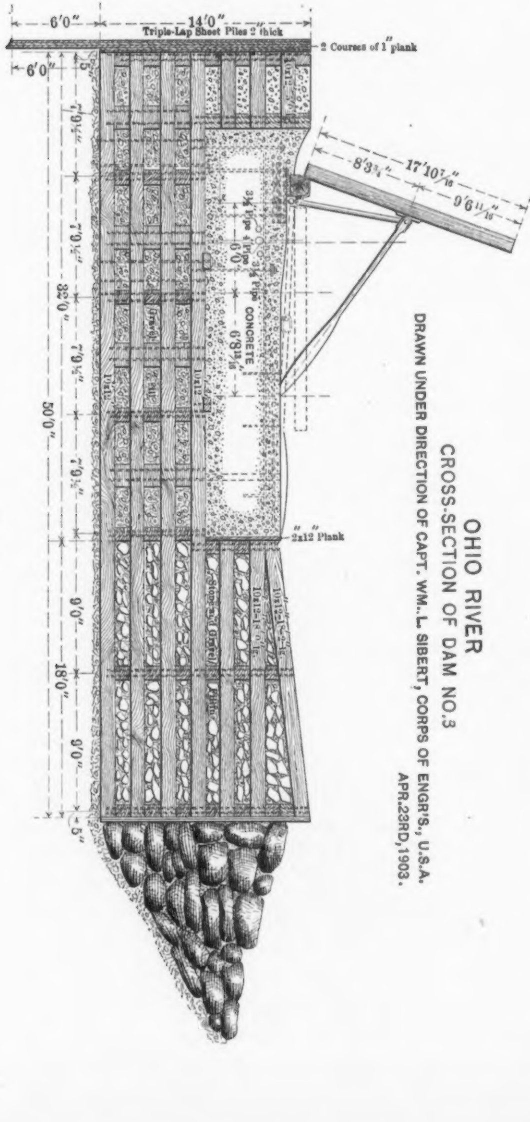

OHIO RIVER
CROSS-SECTION OF DAM NO.3
DRAWN UNDER DIRECTION OF CAPT. WM. L. SIBERT, CORPS OF ENGR'S., U.S.A.
APR. 23RD, 1903.
Triple-Lap Sheet Piles 2" thick
2 Courses of 1" plank
2x12" Plank
CONCRETE
Gravel
Stone and Gravel
6'0"
14'0"
8'3¾"
17'10 7/16"
9'6 11/16"
6'8 13/16"
50'0"
32'0"
18'0"
9'0"
7'9½"
5"

Fig. 2.

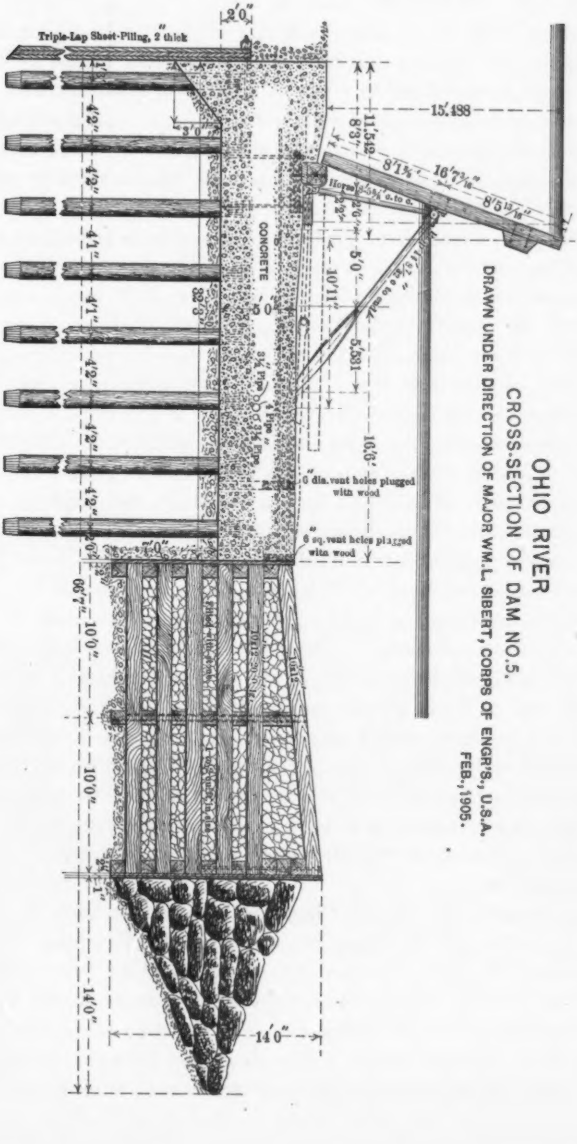

OHIO RIVER
CROSS-SECTION OF DAM NO.5.
DRAWN UNDER DIRECTION OF MAJOR WM.L. SIBERT, CORPS OF ENGR'S., U.S.A.
FEB., 1905.
Triple-Lap Sheet-Piling, 2" thick
CONCRETE
6" dia. vent holes plugged with wood
6" sq. vent holes plugged with wood
Horse
15.488
14'0"

FIG. 8.

The type of foundation shown in Fig. 3 is that used, in the latest foundation work on the river, when rock at practicable elevations does not exist. The water-seal is formed by the wickets, by the up-stream edge of the concrete, and by a row of 9 by 12-in. Wakefield sheet-piles. The concrete is supported on round piles, driven to rock if possible. It is a general principle of construction that the upward pressure beneath a dam should be made as small as possible. This is done by making sure that the up-stream face of the dam is more nearly water-tight than the down-stream face. As there are bound to be leaks between the sheet-piles, an effort has been made in designing these dams to provide free egress for the leakage water in such a way as to insure no movement of the material under the dam. This is done by passageways leading up from underneath the tail of the concrete and opening into the cribs. In order to permit of inspection of the condition of the foundation beneath the dam, vertical holes, 6 in. in diameter, were left in the concrete near the down-stream edge, at intervals of 8 ft. from center to center. These were plugged with swelled wood. The plugs can be removed, or holes can be bored through them and soundings taken to ascertain the conditions existing underneath the dam. In case of scour, the opening can be filled with grout or concrete.

At some of the dams, a wooden covering was placed on top of the foundation concrete, the timbers being anchored to the concrete. The idea was to prevent damage to wickets by interposing an elastic cushion between them and the concrete. It will soon be necessary to repair the wooden top at Davis Island, and it will be a hard task. The wear at the top has been caused very largely by the props running out of the hurters and gouging holes in the wood. It might be advisable in designing the foundation for a movable dam of the Chanoine type to arrange the top surface so as to fit a movable coffer-dam which could be used in replacing movable parts, making minor repairs to the foundation, etc.

The wickets used in the Chanoine dams on the Ohio River are 3 ft. 9 in. wide, and the largest are approximately 18 ft. long. As they are placed 4 ft. apart from center to center, there are 3-in. spaces between the wickets, which serve to prevent interference of the wickets, due to an oblique pull in raising, etc. The discharge of the Ohio River is sufficient to permit of this arrangement, and when the discharge is very small, wooden needles or joint covers are used to cover the spaces

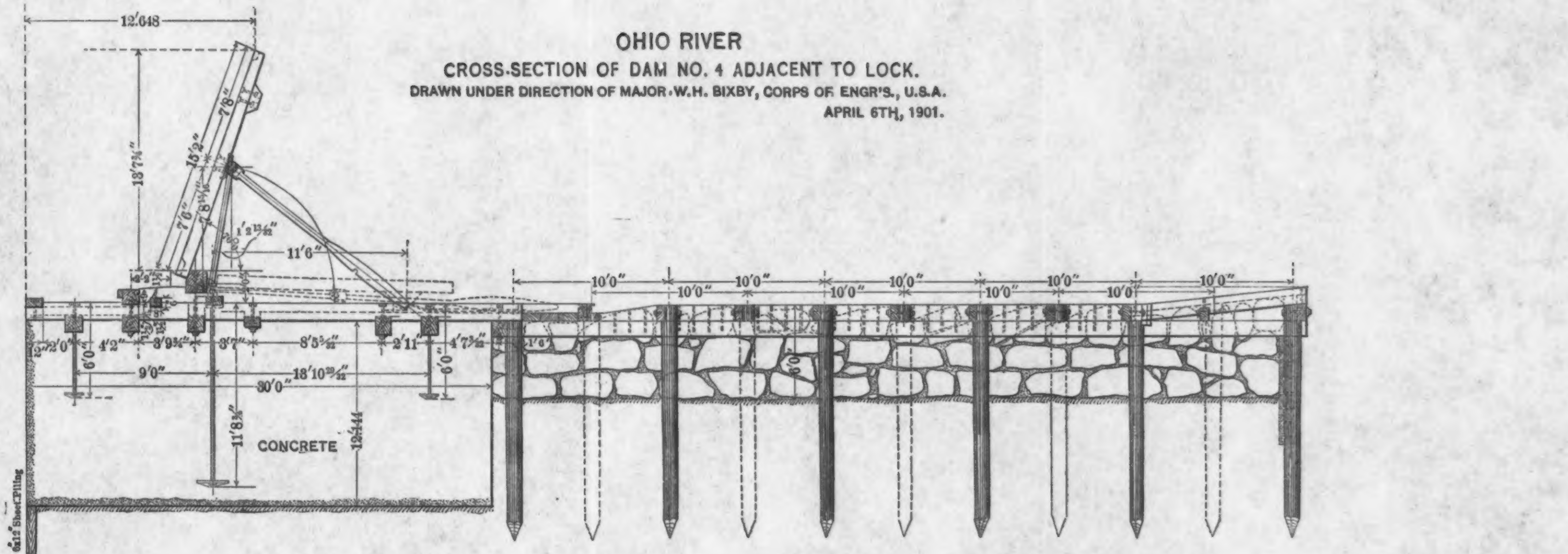
PLATE XXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXIII, No. 1104.
SIBERT ON
IMPROVEMENT OF THE OHIO RIVER.

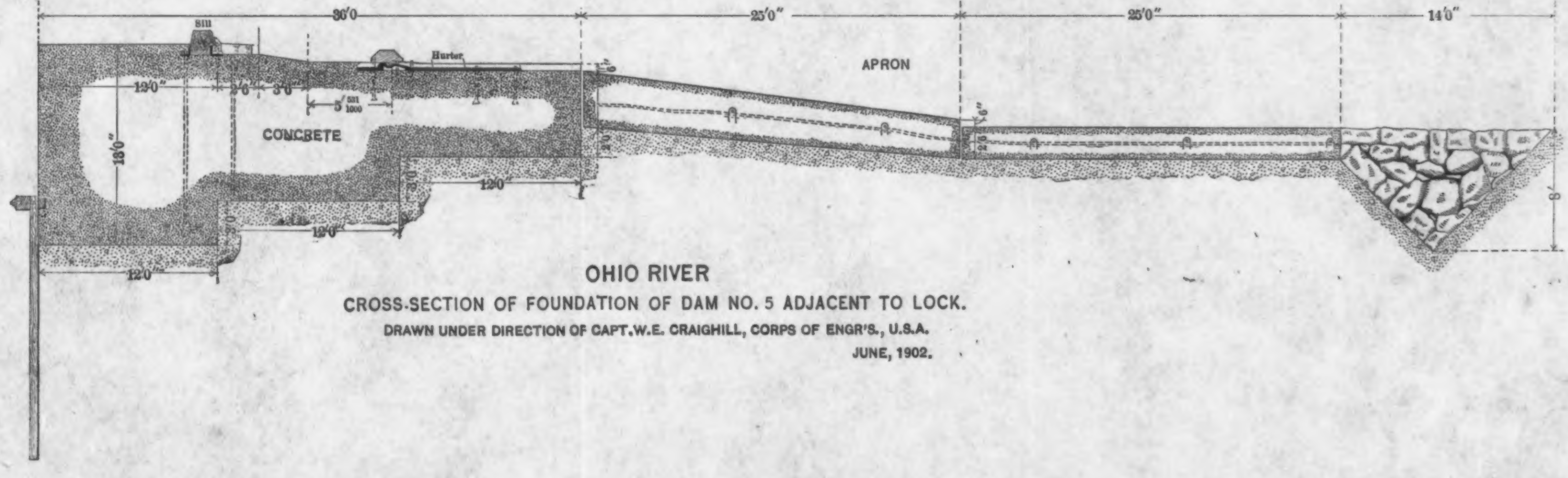
OHIO RIVER

CROSS-SECTION OF DAM NO. 4 ADJACENT TO LOCK.

DRAWN UNDER DIRECTION OF MAJOR W.H. BIXBY, CORPS OF ENGR'S., U.S.A.

APRIL 6TH, 1901.





OHIO RIVER

CROSS-SECTION OF FOUNDATION OF DAM NO. 5 ADJACENT TO LOCK.

DRAWN UNDER DIRECTION OF CAPT. W.E. CRAIGHILL, CORPS OF ENGR'S., U.S.A.

JUNE, 1902.

between the wickets. The wearing away of the wickets has sometimes made it impossible to stop all leakage at the Davis Island Dam, even when the needles are used, and, at times, straw and willows have been used to decrease the leakage. This pool, however, has been maintained for years with no pool below, when leakage was greater than it will be hereafter, this part of the system having been completed.

The pass wickets should be designed so that they will not swing, at least until the upper pool rises to a height of 2 ft. above the crest of the dam. The water has been allowed to rise that high at the Chanoine Dam in the Allegheny River; however, that was under exceptional circumstances. It is generally necessary to commence to lower the pass by the time the upper pool is 1 ft. above its normal level. In order to assist the wicket in maintaining its upright position, cast-iron counterweights are placed in the breach of the wicket. The desirability of making the pass wickets non-automatic until a certain stage is reached, puts an inferior limit on the distance from the foot of the wicket to the journal boxes, and the economy of keeping the length of the horse and prop small provides the superior limit.

It is customary, in the operation of a Chanoine dam in the Ohio, to leave one wicket of the pass down near the river wall of the lock. This affords a passage for small drift which collects at times near the lock wall.

The horses have a large factor of safety when the normal tension to which they are subjected is considered; but very often they have been damaged by being struck by boats, snags, etc. Those in the dams described generally have been provided with side bars 1½ by 3 in. in section. The latest ones have 2 by 4-in. side bars. At first, the axles were made of cast steel, but many were broken, and forged steel axles were then adopted.

The props have been 3½ to 4 in. in diameter, with a large excess of material at the lower end, in order to ensure that they will remain in position in the hurter channel when the dam is being raised. The eddy currents have a tendency to swing the props sidewise, and to prevent their seating properly against the step in the hurter. It has been considered by some that the large bulge at the bottom end of the prop contributes to this by offering a greater surface for the water to act against, and of late experiments have been made with cylindrical props, 5½ in. in diameter throughout their length. It sometimes happens in

lowering the wickets that the props do not run back properly on the hurter grooves, but catch and prevent the wicket from falling. It is then customary to lift the lower end of the prop with a hook on the end of a line from the maneuvering-boat derrick, this hook engaging in a notch in the lower part of the prop.

The current increases greatly as the last few wickets are reached in raising the dam, and these wickets should be provided with heavier props than the other wickets of the pass. One of the greatest troubles encountered in raising the pass is the difficulty of catching hold of the handle plates of the last few wickets to be raised. It is believed that the wickets of the pass nearest to the first pier might well be provided with chains reaching from the bottom handle plate of one wicket to the top handle plate of the next wicket on the side toward the lock. The last few wickets can then be raised by these chains.

The Bear-Traps.

A bear-trap weir was installed at Davis Island in 1889. The gates were of wood, and were 52 ft. long, forming a weir of the type known as the "old bear-trap." It was intended for use as an automatic weir for the regulation of the pool levels, and also as a drift-pass. This bear-trap weir proved to be of much assistance in operating the dam. After the original bear-trap at Davis Island had been in service for about sixteen years, it was replaced by a new bear-trap, the leaves being of wood heavily bound with steel, and proportioned on the basis of 100 upward lifting force to 66 downward.

These new leaves are much stronger and more rigid than the old ones, and work very satisfactorily. The thought in the design was to build the leaves so that they would just about float, and thus eliminate the need of air. For lengths not exceeding 60 ft., the plan is very promising.

At Dam No. 6 two bear-traps were installed, each being 120 ft. long. The principal members were of steel filled in with wood. The leaves were proportioned on the basis of 100 lifting force to 80 downward. In order to assist in the raising of the gates, arrangements were made for increasing the buoyancy by air forced to the under side of the lower leaves. The air was intended to be pocketed between the ribs of the girders. This proved to be of little value, because the air could not be retained where it was needed. If both bear-traps are up and one is

FIG. 1.—BEAR-TRAP DAM, LOCK NO. 5, OHIO RIVER.

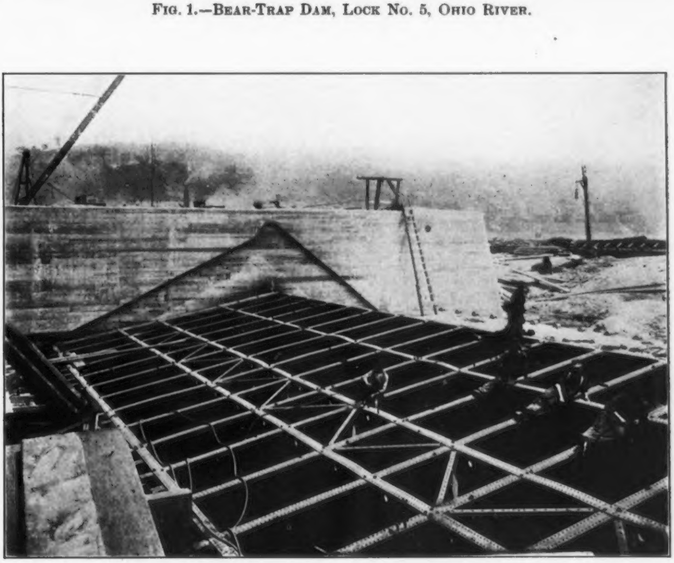


FIG. 2.—BEAR-TRAP DAM, LOCK NO. 5, OHIO RIVER.

lowered temporarily, air is not used in raising it again. It takes a head of about 2½ to 3 ft. to raise the gates if air is used, and about 3½ ft. without it. These gates rise in from 6 to 8 min., if under a 9-ft. head, and can be lowered in from 3 to 4 min. So much of this dam is of the automatic type—about 25% of the entire dam—that it is difficult to secure the head needed to raise the gates if both are down.

Each of the lower leaves of the bear-traps in Dams Nos. 3, 4, and 5, consists of 19 girders, spaced 5 ft. apart, center to center. There are also five longitudinal braces, called girts, which divide the leaf into 72 sections. Certain girders are solid throughout, and divide the leaf into three water-tight compartments. All the girders are hinged, at the down-stream end, to steel castings anchored to the foundation. The up-stream ends are provided with rollers which support the girders of the upper leaves as they slide over the upper end of the lower leaves. Each of the steel castings, which holds the rollers, terminates in a double hook which engages in stops on the upper end of the up-stream leaf, when both leaves reach the limiting height. In order to hold the gates in the upright position during repairs, etc., locking-pin holes are provided. The skin-plating of the lower leaf consists of buckle plates and flat plates. Manholes are left in the plates, those in the upper side of the leaf, being provided with covers, are intended for use only in cleaning the leaf, making repairs, etc., while those in the lower side are intended to permit the water in the leaf to be displaced by air. The lower leaf when filled with air is calculated to have such buoyancy as will cause the entire bear-trap to rise in still water, no hydraulic lifting force being needed. The air enters the pockets of the lower leaf through special ball and socket pipe joints.

Leakage from beneath the gates, between them and the piers, is prevented as far as possible by water-seal arrangements placed so as to be pressed against the sides of the piers. At the lower end of the gates, curved plates attached to the leaves move close to similar plates attached to the hinge castings. (Plates XXVI and XXVII.)

The question of leakage prevention is of great importance, and should be carefully considered. The bear-trap gates at the Herr Island Dam in the Allegheny River could not be raised automatically until the space between the upper and lower leaves was decreased by thin boards. They can now be raised under a 4½-ft. head without the use of air.

The upper leaves of the bear-traps at Dams Nos. 3, 4, and 5, consist of a series of nineteen I-beams, spaced 5 ft. apart, center to center. These girders are separated by longitudinal braces. Each I-beam is hinged at the up-stream end to a steel casting anchored to the foundation. The leaf is sheathed with dressed white oak, carefully fitted between the I-beams. This leaf does not have to support any water pressure, and is made as described so as to decrease its weight.

The water for operating these gates is taken from the sides of Piers Nos. 1 and 3 farthest from the bear-traps. This is because there is so much fall around the heads of these piers, and it is important to use all the upward pressure that can be obtained. The conduits are built so that the water enters each bear-trap from each end. This was done in order to decrease the tendency of the leaves to warp. These traps are proportioned on the basis of 100 upward lifting force to 60 downward. They have not yet had sufficient trial to permit of the expression of opinion as to their merits. The upper end of the foundation is arranged for supporting a needle-dam, and this can be used to secure the head needed to raise the gates, if impossible to raise them otherwise, and can serve as a means of holding the pool while they are being repaired.

Chanoine Weirs.

The Chanoine weirs, which are located between the bear-traps and the abutments of all these dams except Dam No. 6, are similar to the passes. The wickets are raised, however, by a winch running on a structural-steel service-bridge, standing on the weir foundation above the wickets.

It was originally intended that the entire dam at Davis Island should be operated from a service-bridge, but before its completion it was decided to place the journal boxes for the bridge in the pass foundations, but not to erect the bridge except above the three weirs. The pass was operated from a small maneuvering boat. During a sudden freshet in 1887, the bridge of Weir No. 1 was destroyed by drift, and, in 1889, the bridge of Weir No. 2 was destroyed in the same way. On a later occasion, the bridge above Weir No. 3 was struck by a coal barge during a freshet, and partially wrecked.

The original trestles were composed of rather light members, and those built later have been made much stronger. It seems to be ad-

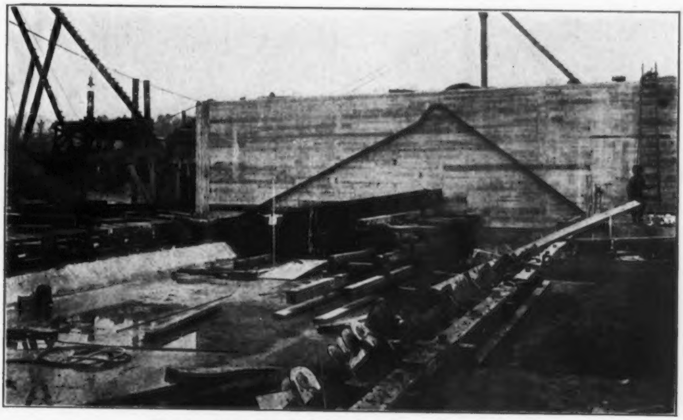


FIG. 1.—BEAR-TRAP DAM, LOCK NO. 5, OHIO RIVER.

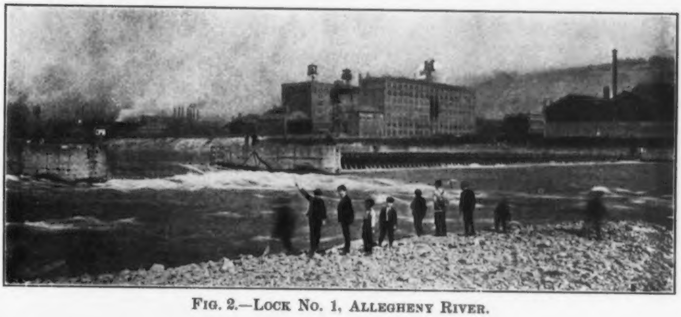


FIG. 2.—LOCK NO. 1, ALLEGHENY RIVER.

FIG. 3.—LOCK NO. 1, OHIO RIVER.

visable, however, to avoid the use of service-bridges in the navigable passes of the Ohio River for the following reasons:

(1).—The bridge and its foundations constitute an extra expense.

(2).—Likelihood of destruction of the bridge by drift or ice.

(3).—The bridge retards the lowering of the dam, because, after the wickets are lowered, it is still necessary to lower the bridge. The lowering of the pass can be effected quickly by a maneuvering boat.

The latest weir service-bridges for these dams have trestles 8 ft. apart, center to center, the trestles being arranged to rotate about their bottom axles, which turn in journal boxes anchored to the foundation. The down-stream journal box is open on top, the axle being prevented from coming out by an iron key. When raised, the trestles are kept in place by steel floor-panels, each hinged at one end to the top of a trestle, and held at the other end by pins attached to the adjoining trestle, the pins passing through holes in castings attached to the end of the floor-apron. The floors form a track for the winch used to operate the wickets, and serve also as a footbridge for the dam-tenders.

An effort was made to make these bridges strong enough to resist the destructive effect of drift, and they were also made strong enough to support the full upper-pool pressure, assumed to be borne by needles resting at the bottom against the oak sill in which the upper journal boxes are embedded, and at the top against horizontal beams resting against the tops of the trestles.

A rolling, hand-operated winch has always been used to maneuver the wickets at Davis Island Dam. The winch is provided with a drum parallel to the crest of the dam. Another winch having a drum parallel to the direction of the current, is used to raise and lower the bridge trestles successively.

The operating winches for Dams Nos. 2 and 5 are to be operated by compressed air, furnished from the power-house through a 4-in. pipe, embedded in the foundation of the dam and ending at the abutment.

The maneuvering boats used to operate the wickets of the passes are provided with derricks, hoisting engines, steam capstans, etc. The hull of one of the latest of these boats is 70 ft. long, by 22 ft. wide, and 3 ft. deep, and is built of wood throughout. A stiff-legged derrick was placed near the bow of the boat; this is operated by a three-drum hoisting engine.

The wickets are raised by a wire line leading from the hoisting

engine around a sheave held at the end of a structural-steel beam, which projects from the bow of the boat. The wire line is fastened to an iron hook at the end of a long pole, which is used to guide the hook until it engages in the handle of the wicket. After the prop is seated in the hurter, the wicket is assisted in seating itself against the sill by pike poles in the hands of the dam-tenders. In lowering the wickets, the upper ends are pulled slightly up stream, until the prop unseats itself, when the wicket is allowed to fall upon the foundation behind the sill.

The boat is prevented from touching the wickets above their axes of rotation by spuds which bear against the wickets below the tops of the horses. The current around the end of the wickets already raised is very swift. A manila rope, of 2 in. diameter, is fastened at one end to the river wall of the lock, and, at the other end, it is wound around the steam capstan on the stern of the boat. This rope prevents the boat from being swept around the ends of the wickets, and also serves to draw the boat back to the river wall after all the wickets have been raised.

The abutments of these dams are not subjected to the severe conditions which exist at the usual fixed dams, because the lifts are smaller; and when a freshet comes, the dams are lowered, thus restoring approximately the normal discharge area of a stream. However, it is necessary to design the abutments so that they will be free from danger of failure, due especially to eddies produced by the flow of water through the bear-traps and by the current of water flowing through the weirs. If the abutments are not founded on rock, they rest on bearing piles, and are protected, not only by a row of Wakefield sheet-piling, but also, below the dam, by stone-filled cribs placed in front of the sheet-piling. The abutments are of the U-type, with a wall extending down stream, the function of which is to prevent the washing away of the bank below the abutment. It is deemed necessary to carry this wall for a distance of from 100 to 200 ft. below the crest of the dam, depending on the character of bank material, value of property, proximity of buildings, etc.

The operation of such a system of movable dams as that described, demands a very careful watch of the discharge of the streams upon the head-waters. This is provided for by reports received daily, or more often if necessary, from observers stationed at various points. It will probably be necessary to construct a Government telephone or telegraph

line along the river, as has been done along the canalized Great Kanawha. Any increase in the low-water flow beyond that necessary to furnish water needed for evaporation and operation, adds to the difficulties of the operation of such a system.

In operating a single dam, there are many things to be contended with. At Davis Island, boats have been known to hit a pier, a bear-trap gate, and lock-gates; there have also been many collisions with wickets. The fact that a floating coal barge destroyed a portion of the service-bridge has been noted. However, if enough spare wickets, horses, and props are kept on hand, those injured can be replaced by a diver without much difficulty. Some of the movable parts were broken by the stern wheels of steamboats before the dams were in operation, and it is very necessary to provide a sufficient number of spare parts. Notwithstanding all difficulties, Davis Island Dam has been operated successfully for twenty-two years, and Dam No. 6 for three years.

The 150 pass wickets of Dam No. 6 have been raised in 1 hour 40 min., the river being at a natural stage of 6.4 ft., and the dam has frequently been raised in 2½ hours. This dam can easily be lowered in 1 hour. (Fig. 3, Plate XXVII.)

DISCUSSION

Mr. Ripley. THERON M. RIPLEY, ASSOC. M. AM. SOC. C. E. (by letter).—A reading of Major Sibert's paper brings again the query which has come to mind many times in the past few years, viz., on what data and after how careful consideration of the questions has been based the assumption that a possible 11 ft. is the maximum which should be provided for on the Ohio improvement, and if 9 ft. is economically necessary at Pittsburg at present, would not 11 ft. or more be the economical development below Portsmouth or Cincinnati?

This query is not an insinuation as to the paucity of data or lack of study of the scheme for the Ohio River as a river, but in its relation to the possibilities and probabilities of contiguous improvements and their bearing on the Ohio River work.

The State of Ohio contains at least one, and maybe two, routes along which it is possible to construct a canal from Lake Erie to the Ohio River with a depth of water of not less than 12 ft.

For several years a determined effort has been made (and is now being made) by some of the State's best men to have such a canal constructed. There are those who believe that the State could expend no money which would be of greater economic benefit than in building such a canal.

Ohio is geographically situated directly between the immense ore deposits of Northern Michigan and the no less immense deposits of coal in West Virginia. Already traversed by the trunk lines of nearly all the railroads from the Atlantic to St. Louis and northern points, what more natural than that some of her citizens should believe in bringing this coal and iron together by the cheapest method, and shipping her manufactured product by her own waterway and the Barge Canal of New York State to New York City, or by her railroads to Atlantic and inland points farther south, or by her canal and river to New Orleans and intermediate points?

Questions such as these should be taken into consideration in any development for the Ohio, as any structures in that river will determine the economic navigable depth of connecting waterways above, and may assist or destroy their usefulness. In fact, a less depth below Portsmouth or Cincinnati than that possible across the State of Ohio might prevent the building of an Ohio Canal, and in any event would be a serious handicap thereto.

Mr. Sibert. WILLIAM L. SIBERT, M. AM. SOC. C. E. (by letter).—The following query by Mr. Ripley is very pertinent:

"On what data and after how careful consideration of the questions has been based the assumption that a possible 11 ft. is the maximum which should be provided for on the Ohio improvement, and if 9 ft. is economically necessary at Pittsburg at present, would not 11 ft. or more be the economical development below Portsmouth or Cincinnati?"

The Ohio and Mississippi Rivers were considered as one system in the project for the Ohio River. At present a low-water depth of 9 ft. is maintained between Cairo and New Orleans by hydraulic dredges. With an actual depth of 9 ft. at low water, a fleet of towboats and barges drawing more than 6 to 6½ ft., would not attempt to navigate the Mississippi River. This is due to the narrowness of the channels at the crossings, and to danger from logs and snags embedded in the bottom and banks of this stream. The draft of single boats would approach more closely to 9 ft. Mr. Sibert.

The practicability of further improvement of the Mississippi River is now under consideration. There are those, whose experience causes their opinion to be of value, who think that it is practicable to maintain an open-channel depth of 14 ft. at low water in the Mississippi between Cairo and New Orleans by dredging and bank protection.

A slack-water depth of 11 ft. in the Ohio River would practically provide for towboat navigation of the same draft as would a 14-ft. open-channel depth at low water in the Mississippi River. In a slack-water system there are no currents to contend with, and the limiting depths and contracted channel widths exist only in the heads of the pools, both of which conditions disappear after leaving the locks.

The 9-ft. depth in the approved project for the Ohio corresponds therefore to the low-water depth at present maintained in the Mississippi for packets, and to a 12-ft. low-water depth for towboats. The Mississippi is naturally at or above a 12-ft. stage from Cairo to New Orleans for a large portion of the year.

Considering the Ohio and Mississippi Rivers then as a trunk line of a great water-transportation system, these would be the reasons for the projected maximum depths in the Ohio River part of it, if such maximum depths were not fixed by other considerations. In the Ohio itself such considerations are cost, hindrances to navigation by many locks, and the practicable height of movable dams across great streams.

The maximum attainable depth on the tributary streams is also a determining factor as to depth in the trunk-line stream. Such tributary streams constitute the feeder lines, and they should be of the same gauge, or the same navigable depth, or, at least, should be of such depth as to permit the transportation of material from such feeder to its market economically, and without transferring the cargo.

Many of these tributaries can only be properly improved with locks and fixed dams, their steep slopes making open-channel navigation impracticable, thus excluding movable dams, were they not otherwise excluded by the certainty of sudden floods accompanied by ice or drift.

Nearly all these streams transport considerable silt, and any slack-water plan for the improvement of silt-bearing streams which involves deep dredging in the upper ends of the pools must take into consideration excessive dredging in maintenance. Fixed dams being neces-

Mr. Sibert. sary, and dredging eliminated, it is seen that such dams must be high or very close together, if depths of as much as 9 ft. are to be made in the upper ends of pools in the tributary streams of the Ohio. Increase of flood damage soon limits the height of fixed dams, leaving out of consideration the fact that such increase decreases the flood velocity of currents and may cause the silting up of pools. Fixed dams, however, can be built higher than one would think, without investigation, and not materially change flood heights. A few inches fall at the dam in extreme flood so increases the discharging capacity as to enable such flood to pass the contracted section with only the few inches increase of height near the dam.

As to assembling material for manufacture, such as coal and iron ore, on the Ohio River, and transporting the iron ore from the Great Lakes *via* a canal and the coal *via* the Ohio River and some of its tributaries, the maximum practically attainable depth in the tributary from which the coal comes would probably fix the maximum needed depth in the Ohio, as far as coal is concerned. Coal in fleets can be transported on the Ohio on a 9-ft. slack-water depth at a cost of about ½ mill per ton-mile.

Increase of cost of transportation, as depth decreases, is more rapid in contracted channels, such as canals, than in open rivers, where increased cargo can be obtained by covering more area with the fleet. This leads to the general conclusion that where a large freight movement is certain the depth in a canal should be governed largely by the available water supply. Should a canal be built connecting the Great Lakes with the Ohio River, the river could be improved to the canal depth, if practicable, for such a distance as to reach a suitable place for the assembling of material for manufacture and the distribution by rail of finished products. The shipment of the manufactured products to New Orleans would be by towboat, with barges of such draft as the improved Mississippi would fix, and which, it is thought, would not exceed the maximum stated for the Ohio.

A depth of 11 ft. on miter sills has been adopted, it is understood, in the Barge Canal of New York. This depth can be obtained in the Ohio, under the 9-ft. project, by a small amount of dredging in the upper ends of the pools, the depth of water on the miter sills being 11 ft.

Unless there were special reasons for a greater depth than about 12 ft. in a canal from the Great Lakes to the Ohio—such as the proximity of suitable and extensive manufacturing sites to the Ohio River terminus of such a canal—the depth of water in the Barge Canal of New York and the maximum obtainable towboat draft in the Ohio-Mississippi would be strong arguments for a standard depth of 11 ft. on miter sills in a canal connecting the Great Lakes and the Ohio.

MEMOIRS OF DECEASED MEMBERS.

WILLIAM BEVERLY CHASE, M. Am. Soc. C. E.*

Died October 26th, 1908.

William Beverly Chase, the son of Levi W. and Harriet Vining Chase, was born in Marengo, Morrow County, Ohio, on November 21st, 1852. His family was of Colonial ancestry, his great-grandfather, Beverly Chase, having served as a New York Militiaman in the War of the Revolution.

His father was an architect and master builder of the old school, and, in his early years, the son lived in the atmosphere of the builder and the artist and became acquainted with the use of the draftsman's tools.

His early education was secured in the high schools provided in the Central Ohio towns of that period, and has been described as "consisting mainly in digging out things for himself," and, to his praise be it said, that during his after life he made most successful use of the ability thus acquired.

In early manhood he removed with his parents to Southwestern Minnesota, where for five years he was engaged in surveys and in acquiring practical experience in the design and construction of railroad bridges.

In 1877 Mr. Chase removed to Oregon, and for more than thirty years was identified with the growth and development of that State. For the first three years he was occupied with map work, and was in the employ of local bridge builders, but, in 1880, his opportunity came with the construction of the Northern Pacific Railroad, surveys for which were then in progress. Beginning as a Topographer for a field party, he was soon at Headquarters in Portland preparing designs and plans for the large number of structures required for the Western Divisions of the railway, then under the charge of V. G. Bogue, M. Am. Soc. C. E.

From 1884 to 1885, he was Engineer of Bridges for the Oregon Pacific Railroad, a line crossing from the Coast through the Western portion of the State, of which the late Isaac W. Smith, M. Am. Soc. C. E., was Chief Engineer. From 1885 to 1890 he was engaged in hydraulic and sanitary surveys and work and in bridge construction for some of the towns of the State, among which were Corvallis and Eugene.

In 1891 Mr. Chase made surveys and designs for a sewer system for East Portland, and, from 1891 to 1895, he was Engineer of Bridges for the Portland Bridge Commission, during which time he con-

* Memoir prepared by D. D. Clarke, M. Am. Soc. C. E.

structed the Burnside Street Bridge, crossing the Willamette River, costing $300 000, and a steam ferry-boat for North Portland. From 1894 to 1896 he was engaged in general practice, while from 1896 to 1902 he was in the service of the City, first as Superintendent of Streets and later as City Engineer, retaining the latter position until, by a change in political parties, the wing with which he had allied himself suffered defeat.

During all these years and until his last illness, he continued to act as Consulting Engineer for the County Commissioners, who under the law are charged with the duty of maintaining all the bridges and ferries crossing the Willamette within the City limits. After the retirement from the service of the City, he was engaged in making surveys and designs for various towns for water supply, street pavements, etc., the Towns of Astoria, Corvallis, McMinnville, Rainier, and Tillamook, Oregon, and North Yakima, Washington, being among the places which he served acceptably.

It was while engaged in making an examination of a water-supply project for the Town of McMinnville, in July, 1908, that Mr. Chase suffered a severe paralytic stroke from which he never recovered, being confined to his bed from that time until his death, which occurred October 26th, 1908, at the Good Samaritan Hospital, Portland.

Mr. Chase was known as a genial gentleman and a man of recognized ability and worth. He was a Christian, from early life, and although not demonstrative he was ever known as a loyal supporter of all things that make for that "righteousness which exalteth the nation." He was long connected with the Centenary Methodist Episcopal Church of Portland, Oregon, and by his loyalty and steadfastness was largely instrumental in sustaining it during a critical period of its history. His zeal in the service of his church was his by inheritance from a godly ancestry.

In 1884 Mr. Chase was married to Miss Georgia Parker of Astoria, Oregon. The death of his wife in 1894, leaving him with their family of three little daughters, and the care of his aged father, greatly modified Mr. Chase's purposes and efforts in his profession, obliging him to put aside congenial opportunities offering exercise of his energies in wider fields, but through all his life he was an inspiring example of what may be accomplished alone, following a natural bent, supplemented by faithful application and courage.

Mr. Chase is survived by two daughters, Misses Marion and Jessie Chase, of Portland, Oregon, and by a brother, the Rev. Charles E. Chase, of San Francisco, and a sister, Mrs. Lucia C. Bell, of Fruitvale, California.

Mr. Chase was elected a Member of the American Society of Civil Engineers on September 6th, 1899.

MARTIN WILLIAM MANSFIELD, M. Am. Soc. C. E.*

Died September 25th, 1908.

Martin William Mansfield was born at Ashland, Ohio, on November 19th, 1850. His father, Martin H. Mansfield, was of English birth; his mother was Anna Saiger, of Mifflin, Pennsylvania.

Mr. Mansfield was graduated from Rensselaer Polytechnic Institute with the Class of 1871. In September of the same year, he entered the service of the Pennsylvania Lines West of Pittsburg as Assistant Engineer in the Maintenance-of-Way Department on the Cincinnati and Muskingum Valley Railroad (a subsidiary line), at Zanesville Ohio. He was promoted successively to Engineer of Maintenance of Way, Superintendent, and Assistant Chief Engineer, which position he held at the time of his death.

On June 24th, 1878, Mr. Mansfield married Miss Carrie Sampsell at Ashland, Ohio, who survives him, with their son, Sampsell W. Mansfield, and daughter, Miss Corinne S. Mansfield.

As a student at Troy Mr. Mansfield was diligent, earnest, and successful. One of his classmates writes of him, "he gave evidence at that time of the unusual talent, that crowned his later years, for working out difficult and abstruse mathematical problems." This talent was indeed characteristic of the man, and was frequently called into play by special lines of investigation assigned to him by his superior officers, who recognized his ability to analyze a mass of apparently heterogeneous facts, reduce them to order, and find the underlying fundamental principle.

Kindly, affable, and accessible, but strict in discipline, he commanded the esteem and good-will of his subordinates. Earnest, conscientious, and upright, in all things, he had the confidence of his superiors. Quiet and unassuming in manner, cheerful in disposition, and equable in temper, he won the respect of all who came in contact with him.

Mr. Mansfield was elected a Member of the American Society of Civil Engineers on July 5th, 1882, and by his death the Society loses one whose professional abilities and private character were an honor to it.

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* Memoir prepared by Thomas H. Johnson, M. Am. Soc. C. E.

MARK WILLIAM SCHOFIELD, M. Am. Soc. C. E.*

DIED NOVEMBER 27TH, 1908.

Mark William Schofield was born in Smithfield, Rhode Island, on November 10th, 1846. His parents were of English stock, and came to the United States about the year 1844. His father died while he was very young, and his mother some years later, in 1864. During his early life he attended the village school in Georgiaville, and later, the Lapham Institute, in Scituate, an advanced academy where many men prominent in later life received a large part of their education.

While attending school his energetic nature led him to spend a portion of his time in the mill in his native town, where he acquired much practical information regarding the details of cotton machinery. His tastes, however, led him toward the profession which he afterward pursued, and early in 1867, and previous to July, 1868, he was for a time with Mr. William S. Haines, and with Cushing and DeWitt, two of the older surveying and engineering firms of Providence. In July, 1868, Mr. Schofield went West, and was engaged on the surveys of the Cairo and Vincennes road, with which General Ambrose E. Burnside was at that time closely identified. Desmond FitzGerald, Past-President, Am. Soc. C. E., was then in charge of the party of which Mr. Schofield was a member.

Late in the fall of 1869 he returned to Rhode Island and re-entered the office of the late Samuel B. Cushing, Sr., M. Am. Soc. C. E., in Providence, where, with the exception of about two years spent on the Northern Pacific Railroad, he remained until the death of Samuel B. Cushing, Jr., M. Am. Soc. C. E., which occurred in 1888. He then carried on the business as the successor of the Cushings, until his death in November, 1908.

In 1869 and 1870 he was leveler on the preliminary survey for the Milford and Lowell Railroad, of which the elder Mr. Cushing was Chief Engineer. In 1873-1874 he was Engineer in charge of the construction of the East Providence branch of the Providence and Worcester Railroad. In the spring of 1881 Mr. Schofield again went West, and, until September, 1882, was Assistant Engineer on the Yellowstone Division of the Northern Pacific Railroad, his section lying between Billings and Miles City, Montana. Here he served with great credit, and, by his faithful and painstaking work, won the confidence and respect of those in the Chief's office. After the completion of the Northern Pacific Railroad, Mr. Schofield resided in Providence and conducted a conservative engineering business of a general nature, doing active work up to within three weeks of the time of his death.

*Memoir prepared by W. H. G. Temple, M. Am. Soc. C. E.

His whole life was marked by that sterling character, unswerving honesty, and strict loyalty to the interests of his clients, which won the respect of all with whom he had either business or social relations, and he unquestionably filled the essential requirements of an honorable man.

Mr. Schofield was married on December 18th, 1873, to Annie S. Brown, a descendant of Chad Brown, one of the early landowners of Providence. His widow, together with four children, survives him.

Mr. Schofield was elected a Member of the American Society of Civil Engineeers on May 1st, 1907.

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